

A Beginner's Guide to the Steel Construction Manual

An introduction to designing steel structures using the
AISC Steel Construction Manual, 14th edition.

By

T. Bart Quimby, P.E., Ph.D., F.ASCE

Owner & Principal Engineer
Quimby & Associates
Eagle River, Alaska

Professor of Civil Engineering
University of Alaska Anchorage

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By T. Bartlett Quimby, P.E., Ph.D., F.ASCE

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Preface

The creation of the Beginner's Guide to the Steel Construction Manual (BGSCM) was prompted by the major rewrite of the AISC specification that appeared in the 13th edition of the Steel Construction Manual (SCM). When textbooks were slow to respond to the change, I started thinking that a web based approach would be more responsive. After further thought the site was expanded to include undergraduate topics in structural engineering for civil engineers.

Initially, my thought was to simply put together my 20 years of lecture notes to supplement the same wonderful textbook that I have been using for the past couple of decades. As the work got to be more extensive than a set of notes, I decided to spend the summer trying to morph this into a stand-alone on-line textbook. The next two summers saw the work expanded to include more example and home problems plus refinements recommended by users of the site.

One of the recommendations was to create a hard copy version of the on-line text. How hard can that be? Well, it turns out that converting the format over has been a very major undertaking. In the process, an in-depth review of the material took place and many unanticipated improvements were made. It was a lot of work.

During the summer of 2011, AISC released the 14th edition of the Steel Construction Manual. After some soul searching, I decided to go for an update to on-line and hard copy texts. With over 20,000 unique visitors per month to the online version, people seem to like it.

While this hard copy version is nice, the CURRENT, and primary version, will always be available free on the web. All the latest changes will be made there and be instantly available. That is the one of the beauties of the web based text. The other is that the online version can contain media types not available in printed texts.

There are a couple of philosophical bases that have driven the particular approach to this work.

First, it is my intent that the student becomes comfortable referring to the base specifications used in design. As a result, if some information is in an industry standard or specification that the students need to know, I refer them to the particular section and let them read it there. This requires that the student have the relevant industry documents nearby whenever reading this text.

Second, it has always bothered me that graduating structural engineering students are commonly unaccustomed to reading real engineering drawings so I have endeavored to present problems (particularly the homework problems) that are in quasi engineering drawing format. It is hoped that students will quickly grasp the concept of reading engineering drawings and feel more comfortable with their first experiences as a design engineer. To facilitate the goal of enhancing a student's ability to read and use engineering drawings, I have provided drawing sets for homework problems that come from real designs.

Third, and related to item two, is that I have chosen to present problems that are in context of a whole structure. I have seen this done by instructors at other institutions, but it is often difficult to find the time to really develop such problems in a way that students can understand

them. As no one structure supplies all types of problems, I have provided several different structures (a building, a tower, and a truss bridge) that cover most of the bases. I have also provided a large set of drawings of random details that are used by the other structures and/or can be used as standalone problems.

Fourth, I wanted the text presented in a way that allows for quick updates and for media types not possible in printed books. This left the Internet website as the logical choice. You will find that many of the figures have hot links that allow for larger views of all or part of the figure. There are also active links to spreadsheets, powerpoint animations, and tutorials that might help a student to better learn how to implement the requirements of the specification. Unfortunately this hard copy version cannot display these other media types. As a result, the text often refers the reader to the web site where these other media types can be utilized.

Fifth, I wanted this to be free to the students. The website version still is. I had toyed with finding a way to put this in a secure website then policing the integrity of the users, but the whole security issue seems tedious and difficult. So, the online version is free for use in teaching structural engineering design. It is copyrighted, however, which means that I'd rather not see my work show up in someone else's book.

Sixth, I want input concerning the content and pedagogy of this text. I have been fortunate to have received several very good suggestions from people that have viewed the online text. I welcome all input concerning the text.

So there you have it. My thoughts behind this work. Any comments about the text are appreciated.

I also am pleased to thank the people that have (and those that will) send in suggestions for improvement to the text. These have already made a substantial difference in the work and I look forward to receiving more. No text of value is written in a vacuum or an ivory tower.

- T. Bart Quimby, P.E, Ph.D.

Owner & Principal Engineer, Quimby & Associates, Eagle River Alaska
Professor of Civil Engineering, University of Alaska Anchorage



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Countless Readers of the site and text—my editors

We constantly receive email from readers with error reports (mostly format), suggestions, criticism, and kudos. All of this feedback is extremely helpful.

Chapter DC – Basic Design Concepts

DC.1 Introduction to Design Theory

This concept of design is not particular to structural steel, however since this course is often a first exposure to detailed design, this section outlines the principles and concepts associated with the design process.

It is of great value for engineers to understand the intricacies of the design process and its impact on final product and those that use it to a depth not permitted in this application. There are several references that are of great value to helping understand the design process and its impact, however the author's favorite is a thought provoking text entitled *To Engineer is Human*, by Henry Petroski.

The Nature of Design

Design can be described as the process of conceptually creating something that does not yet exist. To accomplish this, designers make use of their knowledge of material behavior and/or of processes coupled with their ability to analyze (i.e. predict) the future behavior of their design to meet some specified need. The success of the design is highly dependent on the thoroughness of the designer's knowledge and a clear understanding of the expected behavior of the final product.

The nature of design is such that there is normally more than one **feasible design** (a design that will "do the job"). A design is defined by a set of **design variables** that can be used in analyzing the design. A design is considered to be feasible if it satisfies the performance criteria (or **constraints**) defined for the problem. Out of the available feasible designs, some are "better" than others. Generally the goal of the designer is to find **optimum design**. The optimum design is generally considered to be the one design that best satisfies the criteria for the project. Typically there is some kind of **objective function** that can be computed from the variables that define a design. The value of the objective function is used to compare feasible designs in order to determine the "best" or "optimum" design.

Another significant feature of design is that it is an **iterative** process. When the design process starts, there are a lot of unknowns. Consequently, the designer must make assumptions about various components of the design. As the design progresses, many of the assumptions can be replaced by known facts when decisions are made about previously unknown components. As the assumptions are refined and replaced, the design is reanalyzed repeatedly.

For example, when designing a floor system, the engineer must know the dead loads in order to proceed with the analysis. Unfortunately, at the start of the design process the designer does not know what the system consists and, therefore, cannot accurately compute the loads on the system. To begin the process, the engineer makes assumptions about the loads based on probable loads. Using the assumed loads, the design proceeds and components are selected. As components are selected, the load assumptions can be re-evaluated and the

analysis performed again. This process iterates until the final calculations use refined loads based on the chosen components.

Design Optimization

Since the objective of design is to find the optimum design, the design problem can be characterized as an optimization problem stated as:

Minimize or Maximize $f_{obj}(dv_1, dv_2, dv_3, \dots, dv_n)$

subject to:

- $f_{constraint_1}(dv_1, dv_2, dv_3, \dots, dv_n)$ compared to allowable value
- $f_{constraint_2}(dv_1, dv_2, dv_3, \dots, dv_n)$ compared to allowable value
- $f_{constraint_3}(dv_1, dv_2, dv_3, \dots, dv_n)$ compared to allowable value
-
- $f_{constraint_m}(dv_1, dv_2, dv_3, \dots, dv_n)$ compared to allowable value

Where

- f_{obj} = the objective function
- d_{vi} = the i^{th} design variable
- n = number of design variables
- $f_{constraint_i}$ = the i^{th} constraint function
- m = number of constraint functions

A feasible design is one where all the constraint functions are satisfied. The optimum design is a feasible solution that represents the design with the best objective function value.

For example, let us say that planet "A" gets a message from planet "B" that "B"ites need a magic potion that "A"ites have a soon as possible. If the potion does not arrive in time then all the "B"ites will all die. With several planets with strong gravity fields along the way, there are several paths by which a rocket could reach planet "B" once it is fired from planet "A". The "A"ites must determine the direction and initial velocity for the rocket that will get there the quickest.

In this problem, the objective function is the time that it takes for the rocket to reach planet "B". The only constraint is that the rocket has to hit planet "B". The two design variables are the initial direction and initial velocity.

Let's see how that problem breaks down mathematically.

The objective function, in this case, would be the function that computes the time it takes for a rocket to reach planet "B".

$$f_{obj} = \text{TimeInTransit}(\text{Velocity}, \text{Angle})$$

Design and Allowable Strength Design philosophies as well as the section on Design Fundamentals. These are found on pages of 2-6 and 2-7 of the SCM.

Until AISC introduced the Load and Resistance Factor Design (LRFD) specification in 1986, the design of steel structures was based solely on Allowable Stress Design (ASD) methodologies. The shift to LRFD has not been readily embraced by the profession even though almost all universities shifted to teaching the LRFD specification within ten years of its introduction. It seems that there was not a perceived need by the profession to change methodologies even though there was ample evidence that LRFD produced structures with a more consistent factor of safety.

Timber

LRFD is relatively new to timber. It was explicitly included with ASD in the National Design Specification with the latest edition of the specification.

Concrete

Because of the complexities of analyzing composite sections using working stress method, the much simpler strength approach was easily adopted with it was first introduced. The strength based (LRFD) method has been in use in the concrete specification ACI 318 since the 1970s.

There were two major differences between the two specifications:

1. The comparison of loads to either actual or ultimate strengths and
2. a difference in effective factors of safety.

Actual vs. Ultimate Strength

The first difference between ASD and LRFD, historically, has been that the old **Allowable Stress Design** compared actual and allowable stresses while LRFD compares required strength to actual strengths. The difference between looking at strengths vs. stresses does not present much of a problem since the difference is normally just multiplying or dividing both sides of the limit state inequalities by a section property, depending on which way you are going. In fact, the new AISC **Allowable Strength Design** (ASD), which replaces the old allowable stress design, has now switched the old stress based terminology to a strength based terminology, virtually eliminating this difference between the philosophies.

**Figure DC.5.1
Comparison of LRFD/ASD Capacities
On a Load vs. Displacement Diagram**

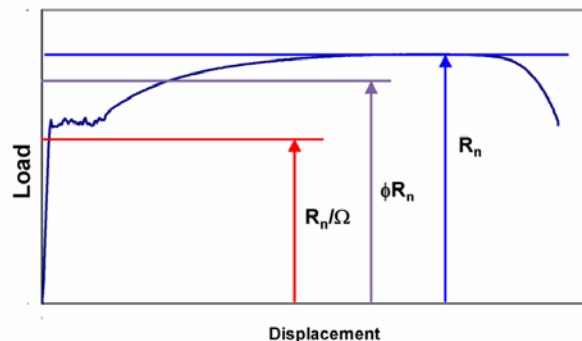


Figure DC.5.1 illustrates the member strength levels computed by the two methods on a typical mild steel load vs. deformation diagram. The combined force levels (P_a , M_a , V_a) for ASD are

Chapter 1 - Introduction

1.1 Introduction

Structural steel is one of the basic materials used by structural engineers. Steel, as a structural material has exceptional strength, stiffness, and ductility properties. As a result of these properties, steel is readily produced in a extensive variety of structural shapes to satisfy a wide range of application needs. The wide spread use of structural steel makes it necessary for structural engineers to be well versed in its properties and uses.

The structural steel industry in the United States is represented principally by the American Society of Steel Construction (AISC). AISC works tirelessly to advance the science and art associated with producing, designing, fabricating, and erecting structural steel. One of their many available resources is the AISC Steel Construction Manual (SCM). This text focuses on training the engineering student to apply the basic design specifications contained in the SCM. In addition, the reader will become familiar with many of the design aids contained in the SCM.

In order for a student to progress through the material presented in this text, it is essential that they are well versed in engineering statics, mechanics, properties of materials, and structural analysis. Some of the required concepts that need to be mastered prior to undertaking this course are:

- Statics
 - The ability to compute reactions on basic structures under given loading.
 - The ability to determine stability and determinacy
 - The ability to determine internal forces in statically determinate structures.
 - develop shear and moment diagrams
 - The ability to solve truss problems (both 2D and 3D) by using
 - method of joints
 - method of sections
 - The ability to solve "machine" problems
 - The ability to compute of section properties including
 - cross sectional area
 - Moments of Inertia for section of homogenous materials
 - Moments of Inertia for composite sections
- Mechanics
 - An understanding of stress and strain concepts
 - The ability to compute stress including
 - axial stress
 - bending stress
 - shear stress (due to both bending and torsion)
 - principle stress
 - stress on arbitrary planes
 - The ability to compute the buckling capacity of columns
 - The ability to compute deflection in beams
 - The ability to compute reactions and internal forces for statically indeterminate structures

- Properties of Materials
 - The ability to read stress-strain diagrams to obtain critical material properties including:
 - Yield stress
 - Ultimate stress
 - Modulus of Elasticity
 - Ductility
 - An understanding of the statistical variation of material properties.
- Structural Analysis
 - An understanding of the nature of loads on structures
 - The ability to compute and use influence diagrams.
 - The ability to solve truss problems (forces and deflections)
 - The ability to solve frame problems (forces and deflections)
 - The ability to use at structural analysis software
- Other Needed Skills
 - The ability to use a word processor
 - The ability to use a spreadsheet program to develop spreadsheet solutions to the problems presented in this text.
 - The ability to use a CAD program to generate drawings and sketches for inclusion in your computations.

After thoroughly studying this text, students will be:

1. able to combine loads to determine design loadings on structures
2. familiar with the AISC Steel Construction Manual
3. able to design steel tension members
4. able to design steel beams
5. able to design composite steel/concrete beams
6. able to design steel columns
7. able to design beam-columns
8. able to design basic bolted connections
9. able to design basic welded connections

At the successful completion of this text, you will be prepared for study addressing more advanced topics such as structural system design, the latest in connection design, and other building code requirements related to the design of steel structures.

1.2 History

It is important that the engineer have a historical perspective on the materials that he/she will be using. This understanding, particularly on the local history of the region(s) they practice, will significantly assist them in applying materials effectively in their designs. The development of the building codes that dictate the application of materials is heavily influenced by the history of the success and/or failures of structures containing that material. Understanding that history will help you to appreciate the requirements that you are subjected to when using a particular material. This will keep you from making the same mistakes of those that have preceded you.

Most published textbooks have a better treatment of the history of steel engineering than is presented here. Eventually, we will add more. If there is someone out there who would like to contribute, please contact me!

A few historical events that are significant to the engineering designing with structural steel are:

- 250 BC: Iron/metal beams used in construction in India. May have been first use of metal members in construction.
- 1777-1779: Cast iron used to construct a bridge in England.
- 1780 the process of rolling various shapes begins, particularly with wrought iron.
- 1777-1840: Cast iron used for bridges.
- 1820: The rolling process used for making rails.
- 1840: Wrought iron begins to replace cast iron in bridge construction.
- 1855: Bessemer process introduced.
- 1870: The Bessemer converter introduced.
- 1870: The rolling of I shapes begins.
- 1890: Steel replaces wrought iron as the dominate metallic building material
- Early 1900s: Formation of building codes begins, formalizing design process and requirements. Principle design philosophy is based on the concept of allowable stresses (ASD).
- mid 1950s: The concrete industry pioneers the strength based design philosophy
- Early 1970s: First strength based design specification introduced by the concrete industry.
- 1986: AISC introduces the strength based Load and Resistance Factor Design (LRFD) specification.
- 1989: AISC releases what was supposed to be the last Allowable **Stress** Design specification.
- 2006: AISC releases a combined LRFD/ASD (Allowable **Strength** Design) specification that incorporates a method for using ASD level loads with the same specification used for LRFD.

As of August 2011, the on-line encyclopedia, Wikipedia, has some interesting information about the history of steel. You should read it. They have some particularly interesting references at the bottom of the page. You can find a number of good sources on steel history by searching the internet.

1.3 An Overview of the AISC Steel Construction Manual

Since the intent of this text is to enable you to develop an ability to apply the 2010 AISC standard, *Specification for Structural Steel Buildings* (ANSI/AISC 360-10) in the AISC Steel Construction Manual (SCM, Figure 1.3.1), it is essential that you have one readily available. This text will refer to it often! This set of notes is based on the Fourteenth edition (2011) of the SCM.

As you progress through the text, you will become very familiar with much of this manual. For now, we need to introduce you to several of the major sections of the SCM and help you to find the information that you will soon be needing. You will want to follow along the discussion in your copy of the manual.

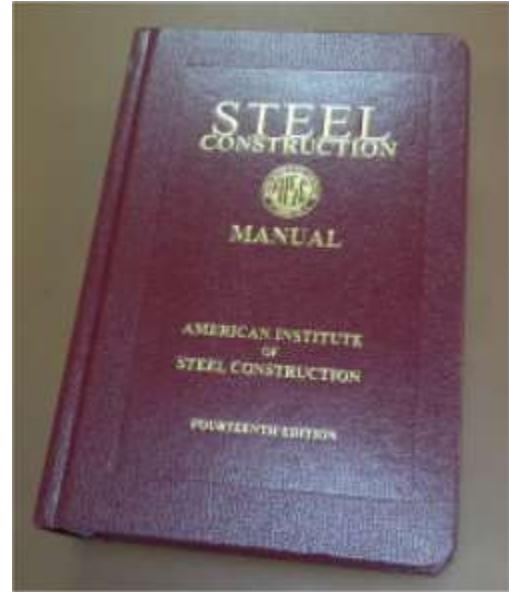
You will undoubtedly note that each of the major sections is tabbed. This makes finding particular section quick and easy. Start by looking at the first tab.

Part 1: Dimensions and Properties

Section 1 is your catalog of available shapes and their section properties. Review the index to this section to see all the shapes that are cataloged here. The shapes listed here are all potentially available. As we will see in Part 2, not all shapes are available in all materials or from all manufacturers.

The first few pages of the section describe the data available for each of the shapes that you find later in the section. You should pay particular attention to the section on standard mill practices. As the SCM points out (and as you doubtless recall from your material properties course) there are unavoidable variation in dimension that result when manufacturing anything. The section on standard mill practices describes where you can learn about the acceptable variations in dimension for the various cross sectional types.

Figure 1.3.1
The SCM



Turn a few more pages into the section and you will find pages and pages of various tables.

The first tables are those for "W" Sections. W sections (sometimes referred to as wide flange sections) include most I shaped sections. The shape designation is given in the left most column and has the form $W\#\#\times\#\#$. The first number is the NOMINAL depth of the member. The second number is the WEIGHT of the section (in units of pounds per lineal foot). For example, a W18x35 section is approximately 18 inches deep and weighs 35 lbs/ft. Note that the sections are sorted by their nominal depth and then by their weights within their depth category. Both sorts start with the largest section first and descend from there.

Look up the W18x35 in the tables. Note that the actual depth is not 18 inches but is somewhat less. While there, notice that they give you two numbers for the depth of the member. The first is a decimal number. The second uses fractions. The numbers are NOT exactly equal. If you read the first page of text in Part 1 of the SCM, you will notice the following statement:

"Tabulated decimal values are appropriate for use in design calculations, whereas fractional values are appropriate for use in detailing." This means that for all your engineering calculations you need to use the decimal values. For any drawings that you produce, the fractional values are sufficient (and often times easier to implement) than the decimal values. This applies to any section property for which two values are given.

You might not understand all of the section properties that are presented. For each section type, a drawing is provided at the top of the page to illustrate where each measurement is taken. The properties derived from the measurements will be explained as we use them. Some should already be familiar to you.

Other Related Information

The AISC website (<http://www.aisc.org/content.aspx?id=2862>) has some other very useful information which will aid you in applying the specification. These include:

- Steel Construction Manual Design Examples. These will help you to see the specification in action and help you as you learn to apply the information.
- Shapes Database. This database comes in a variety of formats which allow you to develop spreadsheets and other applications which can quickly manipulate the data to quickly find efficient designs and/or perform other computations.
- Specifications from 1923 through 2005. These are particularly useful in evaluating older designs or researching changes in the specification over time.

A Note About Units

The steel industry in the United States has traditionally use the U.S. Customary Units (USCU) of feet, inches, pounds, kips, and so forth. The SCM continues in that practice. That said, the manual does provide many values in SI units so that the limit state equations can be used in either set of units. This text will stick with the USCU system since that is what is most convenient in the SCM.

Summary

This manual will need to be your constant companion throughout this text since the text is focused on teaching you to apply the specification in SCM. Take some time at this point to familiarize yourself with it.

1.4 Computational Considerations

When computing the forces on and the strength of structures there is, inherently, a degree of uncertainty and variability. Some sources of uncertainty and variability include:

- *Variation in actual material properties.* For example, you will recall from your laboratory experiences in material properties and knowledge of statistics, there is variation between samples when determining material properties. Structural steel tends to be more homogeneous than other materials so the variability may be less, however, there is still some!
- *Estimated loads.* When determining what loads are to be applied to a structure, estimates are made of the weight of the structure, the magnitude of the live loads that the building is likely to see base on the assumed occupancy of the structure (which may change over time), and the magnitude of environmental loads such as ponding, snow, wind, and seismic. The magnitude estimates of these loads are generally based on probabilistic methods and have been generally accepted by committees of experts as *likely to be sufficient in most cases*. In many complex loading cases, engineers, will make conservative simplifying approximations of how loads are actually applied to structures and/or their components. See *A Beginner's Guide to ASCE -05* at www.bgstructuralengineering.com for more specific information on load calculations.

- *Approximate analysis methods.* All structural analysis techniques are based on theoretical approximations of very complex natural phenomena. This is not to be confused with "approximate methods" taught in most structural analysis courses. Some techniques and methods are more approximate than others.
- *Simplifying assumptions regarding the strength contribution of "non structural" building elements.* This can be considered to be part of the issue under approximate analysis methods. Engineers typically ignore the strength contributions of partitions or other non-structural elements that may indeed add to the strength of a structure. Ignoring the contributions of these elements is generally conservative, except where they actually transmit load to elements that don't have the strength to carry them.

As a result of the uncertainty inherent in structural calculations, it does not make sense to provide extraordinary precision in computations. In all likelihood your engineering calculations may be off by as much 6% from what the actual conditions will finally be. In almost every case, three significant figures (this does NOT mean three decimal places!) will give you accuracy to 1%. This is greater than the accuracy that the computations warrant.

When using electronic computation tools (calculators and computers), it does not make sense to truncated the imprecise digits carried along in the calculations. However, when reporting the results of you calculations, be sure to include only 3-5 significant figures.

Also, you should note that one or two significant figures is not sufficient. For example, the number 0.001 is reported to only one significant figure. This is not enough to know the precision to three significant figures. The reporting should be 0.00100. This is because 0.001 can be interpreted as anything between 0.00050 and 0.001499. This gives a variation of about 50% of the reported value. This level of imprecision is not acceptable.

1.5 Homework Structures

The homework problems all reference one of the drawing sets found on the website for *A Beginner's Guide to Structural Engineering*. The drawing sets are similar in format to structural drawings used in actual practice. The main sets are adapted from actual projects. The miscellaneous details are a mix of problems from actual projects and hypothetical problems that help to reinforce concepts included in the provisions of the SCM and this text.

Four of the drawing sets are used in this text.

- **Dormitory:** A three story building frame that includes both moment and braced frames. This structure was originally designed in the early 1980s but, for a variety of economic reasons, was never built. This structure will provide many of the problems related to beams and beam-columns as well as a few axial force brace members.
- **Tower:** A free standing communication tower that is loosely modeled after a recently constructed (2007) structure. The design has been altered a bit to take advantage of some details found on other similar structures. This structure provides a variety of axial force members and connections.
- **Bridge:** An old truss style bridge built in the mid 1900s. This easily analyzed structure includes a number of axial force and bending members to design.

- **Miscellaneous Steel Details:** This drawing set is the largest of the group and has many standard details as well as details to illustrate specific points.

The drawing sets are downloadable as PDF or AutoCAD drawing files. See the website (www.bgstructuralengineering.com) for details.

specification also has provisions for developing weld procedures for other materials and configurations. In practice, you should consult the AWS specification for selecting the electrodes to be used with your selected materials.

Weld Electrodes

As will be discussed later, we will focus on two primary forms of structural welding: The Shielded Metal Arc Welding (SMAW) and Submerged Arc Welding (SAW) processes. The electrodes used for these two processes are very different in their makeup.

The one thing that you need to know at this point is that part of the designation of an electrode is its specified minimum tensile strength, F_u . The main criteria we will use for selecting weld metals is to find one that somewhat matches the F_u of the base metals.

For the SMAW process, the electrode designation takes the form: E__XX, where the blank contains the F_u of the base metal and the XX is a number of more importance to the welder than the engineer. For example, and E60XX electrode has an F_u of 60 ksi. SMAW process electrodes are available in F_u values of 60, 70, 80, and 100 ksi. Figure 2.2.1 shows the electrode ends that attach to the welding machine for some E70XX electrodes.

For the SAW process only the first number of F_u is used. SAW process electrodes go by the designator F__XX. The common versions include F6XX, F7XX, F8XX, F10XX, F11XX which have F_u values of 60, 70, 80, 100, and 110 ksi respectively.

We will go into electrode selection and use more in the chapter on weld design.

Figure 2.2.1
SMAW Process Electrodes



2.3 Bolts

Bolting is a very common method of fastening steel members. Bolting is particularly cost effective in the field.

The precursor to bolting was riveting. You will probably have occasion to assess connections made with rivets sometime in your career, particularly if you work on restoration projects. Riveting was a very dangerous and time consuming process. It involved heating the rivets to make them malleable then inserting them in hole and flattening the heads on both sides of the connection. The process required an intense heat source and a crew of three or more workers. Figure 2.3.1 shows a riveted connection in a bridge structure.

In the mid 1900s, high strength bolts were introduced and quickly replaced rivets as the preferred method for connecting members together in the field because of their ease of installation and more consistent strengths. High strength is necessary since most bolts are highly tensioned in order to create large clamping forces between the connected elements. They also need lots of bearing and shear strength so as to reduce the number of fasteners needed.

There are two basic ASTM high strength bolt specifications and one non-high strength ASTM bolt specification that we will be using.

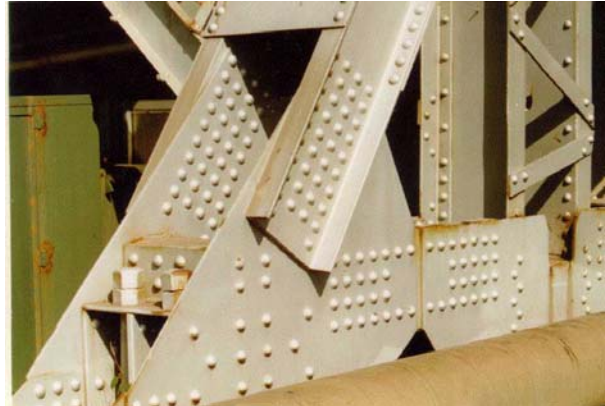
The high strength bolts are ASTM A325 and ASTM A490. The non-high strength bolt is ASTM A307.

New with the SCM 14th addition is the introduction of bolt "Groups" in SCM J3.1 for high strength bolts. ASTM A325 bolts are assigned to "Group A" while ASTM A490 bolts are assigned to "Group B". ASTM A307 bolts are not assigned to a group since they are not considered to be high strength bolts.

The ultimate tensile strengths for A325 and A490 bolts are 120 ksi and 150 ksi respectively. These values are rarely needed in applying the equations in the AISC specification, but are useful to know when using theoretical equations for special circumstances. We will also be obtaining bolt strength information for use in the AISC equations from SCM table J3.2 on page 16.1-120. We'll cover that table in more detail later.

The ASTM bolt specifications require that the bolts and their associated nuts and washers be clearly identified with their specification number. Figure 2.3.2 shows the identifiers for A325 and A490 bolts.

**Figure 2.3.1
A Riveted Connection**



**Figure 2.3.2
Bolt Identifiers**



Chapter 3 – Tension Members

3.1 Tension Member Overview

It is now time to actually learn how to design something! We start with tension members because they are relatively simple. There are only a few limit states to worry about.

Now that we are ready, please turn to SCM Chapter D (SCM page 16.1-26) and let's get started.

Controlling Limit States

In SCM Chapter D we look at three limit states that relate to the member itself. We will then look to SCM Chapter J to look at some connection related limit states that apply to connecting elements and/or end conditions of general tension members.

The SCM Chapter D limit states that we will consider are:

- Slenderness
- Tensile yielding
- Tensile rupture

In SCM Chapter J we will look at the limit states of:

- Tensile yielding of connecting elements
- Tensile rupture of connecting elements
- Bolt bearing on the edge of the bolt hole
- Block shear rupture of strength at end connections of tension members

As discussed in an earlier chapter, limit states represent conditions that limit the usefulness of a member. Generally only one of the limit states will control the design. All must be checked to ensure that your member is adequate for its intended purpose. The following sections present the limit states as implemented by the AISC specification. The example problems show how the limits are applied.

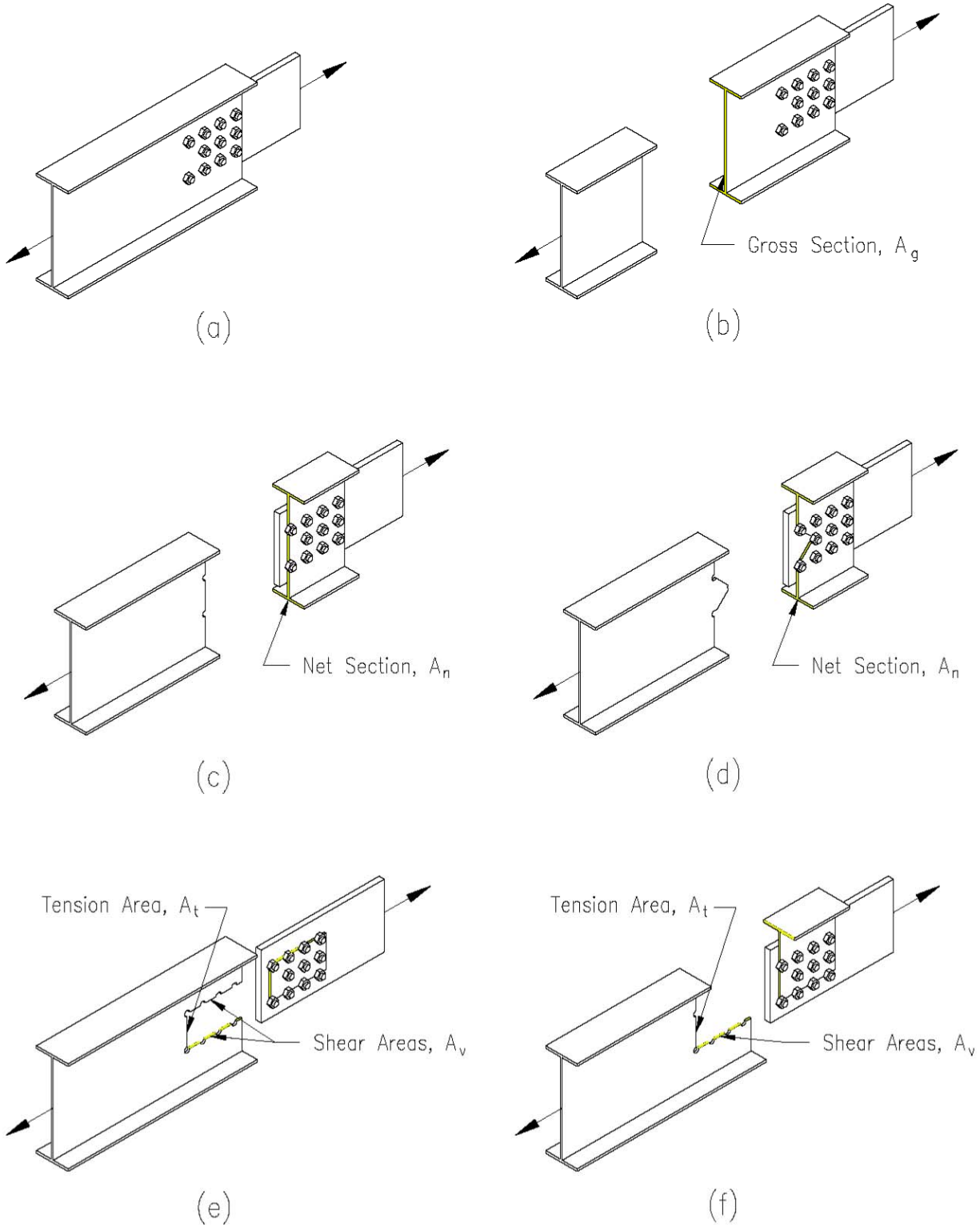
A brief overview of the limit states is presented here.

Figure 3.1.1 illustrates "failures" associated with the various strength based limit states.

Figure 3.1.1(a) shows the end of a W section attached to a short connecting plate. The other drawings in the set illustrate the various strength based failure limit states for the W section.

Note that the connecting plate shown is a separate member and has its own set of limit states that used to define its tensile strength. For this example, we will only focus on the capacity of the W section.

**Figure 3.1.1
Tensile Strength Limit States**



lines in the failure path. **Effective hole diameter** and **failure paths** are two new concepts to be considered.

First, let's consider the effective hole diameter. SCM D3.2 says that "the width of a bolt hole shall be taken as 1/16 in. larger than the *nominal dimension of the hole.*" (emphasis added). A standard hole is 1/16 inch larger than the bolt it is to accommodate. This means that the effective hole diameter, for A_n calculations, is to be taken as 1/8" larger than the bolt (i.e. 1/8" = 1/16" for the actual hole diameter plus an additional 1/16" for damage related to punching or drilling.) So, if you specify 3/4" bolts in standard holes, the effective width of the holes is 7/8" (i.e. 3/4" for the bolt diameter + 1/16" for the hole diameter + 1/16" damage allowance.)

**Figure 3.4.2
Bolt Holes**

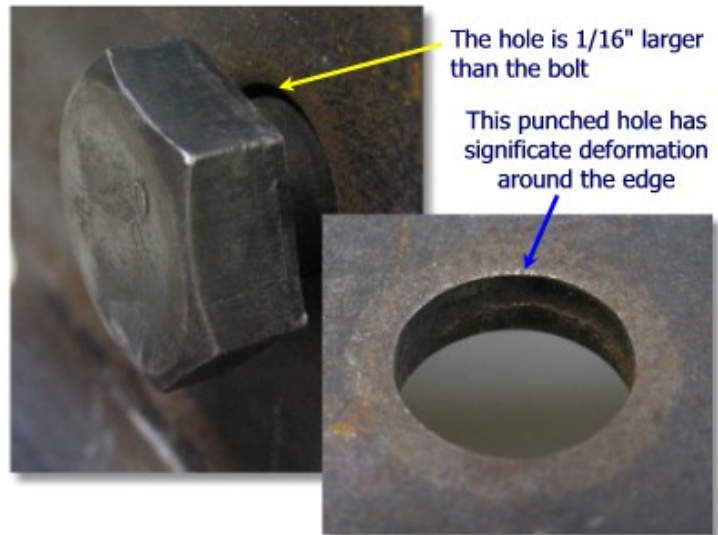
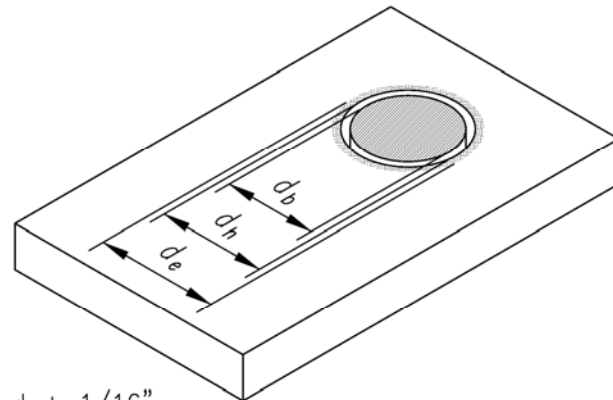


Figure 3.4.2 shows a typical standard hole and the dimensions that are related to it.

The next concept that needs discussing is the concept of failure paths. Failure paths are the approximate locations where a fracture may occur.



$$d_h = d_b + 1/16''$$

$$d_e = d_h + 1/16'' = d_b + 1/8''$$

When considering failure paths, you will always start from the side of the

member and cut perpendicular to the axis of the member to the center of a hole. The path will then extend from center of hole to center of hole until you take a path to the opposite side which is perpendicular to the axis of the member. The failure path will be located so that it sees the maximum force in the member, in other words all the bolts are on the side of the rupture line that is also connected to another member. Any path that leaves bolts on both sides of the rupture is not feasible unless the member you attach to fails simultaneously along the same path... something that cannot happen for a number of reasons.

Another way to look at this problem is that the failure separates the main member from the connection. All the bolts stay with the connection.

The Limit State:

The basic limit state follows the standard form. The statement of the limit states and the associated reduction factor and factor of safety are given here:

LRFD	ASD
$P_u \leq \phi_t R_n$	$P_a \leq R_n / \Omega_t$
Req'd $R_n = P_u / \phi_t \leq R_n$	Req'd $R_n = P_a \Omega_t \leq R_n$
$P_u / (\phi_t R_n) \leq 1.00$	$P_a / (R_n / \Omega_t) \leq 1.00$
$\phi_t = 0.75$	$\Omega_t = 2.00$

The values of P_u and P_a are the LRFD and ASD factored loads, respectively, applied to the member.

Nominal Block Shear Rupture Strength, R_n :

The limit state value computed is the force that STARTS the rupture. Historically, the SCM equations have assumed that rupture initiates either on the tensile area(s) or on the shear area(s), not on both surfaces simultaneously. With the AISC specification found in 13th edition of the SCM, revised equations were introduced. The new equations continue in the 14th edition (Equations J4-5, in SCM J4.3). You should also read the commentary on the section (SCM pg 16.1-411).

The basic concept for computing the block shear rupture strength is that we compute the strength of the tension region and the strength of the shear region and add them together. If you look at SCM equation J4-5 you will see the two terms. SCM equation J4-5 computes the nominal resistance, R_n , or strength associated with block shear.

You should also notice that there are actually two equations in equation J4-5. The term to the left is the situation for shear rupture/tensile rupture and the equation to the right is for shear yielding/tensile rupture. The way the equations are written, you compute both values and take the smaller of the two.

One thing that you might notice right off is the presence of a "0.6" in the shear terms. It turns out that the F_u and F_y values are determined by tension tests. The comparable terms in shear are approximately 0.6 times the values for tension. Consequently, where ever shear strength is computed, we use the tension values multiplied by 0.6. Chances are, whenever you see $0.6F_y$ and $0.6F_u$ in an equation then that equation is computing shear strength.

The other quantity that will be new to you is the U_{bs} quantity. U_{bs} was new with the 13th edition and continues in the 14th edition. This quantity is used to account for non-uniform tensile stress distribution when the block shear region is non-symmetric or in any other case where non-uniform tensile stress is likely to occur on the tensile fracture area. This condition generally occurs whenever there is only one shear area in the failure path. The SCM commentary gives examples of such situations (SCM page 16.1-412). We have this situation in our example problem 3.1.

-SC	This symbol indicates that the connection is considered to be slip critical. This means that the contact (faying) surfaces must meet particular requirements so as to ensure an adequate coefficient of static friction (See SCM pg 7-5 for more detail).
------------	---

For example, an A325-X bolt would be a bolt made from A325 steel and would be installed such that the threads are excluded from any shear plane(s). An A490-SC bolt is a bolt made from A490 steel and would be installing on a slip critical connection where the faying surfaces must meet special requirements.

Limit States

The primary objective of checking all strength based limit states to ensure that the strength of the structural element is strong enough to handle anticipated forces exerted on them. In the case of bolts, this can be expressed as:

$$\text{The FORCE on the bolt} \leq \text{the STRENGTH of the bolt}$$

For bolts the forces applied to the bolts can be resolved into shear (perpendicular to the bolt axis) or tension (parallel to to the bolt axis) components.

Force on the Bolts

The force on any given bolt is the result of the forces being applied to the connection and the geometry of the connection. Principles of Mechanics and Structural Analysis are used to determine the force on any particular bolt in a connection. The next section discusses several commonly used methods for computing the forces on a bolt.

Strength of a Bolt

Bolts have one tensile capacity and two shear capacities. Typically the SCM denotes the nominal capacities of each as R_n . The capacities are defined by limit states.

Tensile capacity is controlled by the tensile rupture limit state which can be stated as:

$$\text{The TENSILE FORCE on the bolt} \leq \text{The TENSILE RUPTURE STRENGTH of the bolt}$$

Shear capacity is controlled by one of two limit states:

- Slip (The force that overcomes the frictional capacity of the connected surfaces)
- Shear Rupture (The force that causes shear failure of the bolt)

For the case of shear capacity, the limit states can be stated as:

$$\text{The SHEAR FORCE on the bolt} \leq \text{The SLIP STRENGTH of the bolt}$$

or

$$\text{The SHEAR FORCE on the bolt} \leq \text{The SHEAR RUPTURE STRENGTH of the bolt}$$

It is common in connections with pins, rivets, and bolts to connect more than two members together with a given fastener. The force transferred by a fastener then becomes a function of the number of shear planes in the connection. As will be seen in the next section, a more convenient, or maybe more appropriate, way of expressing the shear limit states is:

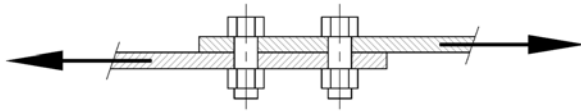
The SHEAR FORCE on a SHEAR PLANE \leq The SLIP STRENGTH of a SHEAR PLANE

or

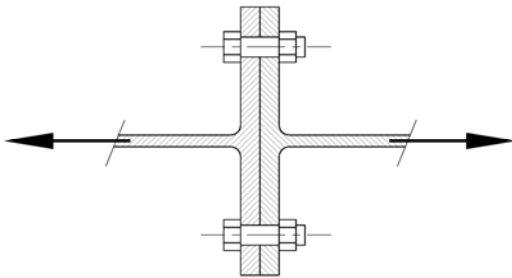
The SHEAR FORCE on a SHEAR PLANE \leq The SHEAR RUPTURE STRENGTH of a SHEAR PLANE

The concept of shear planes is presented in the next section.

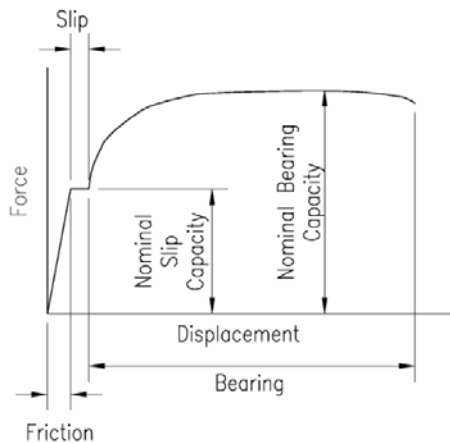
**Figure 4.2.1
Bolts in Shear**



**Figure 4.2.2
Bolts in Tension**



**Figure 4.2.3
Connection Load/Deformation Curve**



friction capacity is the result of the normal force (N) between the plates created by the bolt

4.2 Mechanics of Load Transfer

Mechanical Fasteners (i.e. bolts, rivets, and pins) are most frequently used in structural steel connections where the load direction is perpendicular to the bolt axis as shown in Figure 4.2.1. In this situation the principle force in the bolt is shear.

Less frequently, the bolts are placed such that their axis is parallel to the direction of force as shown in Figure 4.2.2. Here the bolts the principle force in the bolts is tensile.

Let's look at the force transfer mechanisms in more detail.

A Bolt in Shear

As mentioned in the prior section, bolts may be installed as snug tight or fully tensioned, the difference being that the snug tight installation does not provide a significant clamping force between the connected members. Let's begin this discussion by considering a fully tensioned bolt.

If you were to place the connection shown in Figure 4.2.1 in a tension test, the force vs. deformation curve would look something like what is shown in Figure 4.2.3.

As the load is progressively applied to the connection, the major force transfer between the connected plates would be by friction. The

Sample Spreadsheet Computation

This spreadsheet considers both straight slip resistance and combined tension and shear since the modifier due the presence of tension is a simple modifier to the computation for shear capacity.

Bolt Slip Capacity

SCM J3.8&9

Bolt:	A325-N	
A_b	0.4418in	
F_{nt}	90ksi	
μ	0.30	
D_u	1.13	
h_f	1	
T_b	28k	
N_s	1	2 per bolt
N_b	8	8 bolts

Total bolts 16

Total Shear Planes 24

R_{nv} 9.5k/shear plane

R_{nt} 39.8k/bolt

R_{nv} 227.8k/connection

R_{nt} 636.2k/connection

LRFD

Tension, T_u 250k/connection

Shear, V_u 200k/connection

ϕ_v 1

k_s 0.506

$\phi R_{nv} =$ 115.3k/connection

$V_u / \phi R_{nv} =$ 173.4%...NG

ASD

Tension, T_a 200k/connection

Shear, V_a 150k/connection

Ω_v 1.5

k_s 0.407

$R_{nv} / \Omega =$ 61.9k/connection

$V_a / (R_n / \Omega) =$ 242.4%...NG

Check tension: (SCM J3.6)

ϕ_t 0.75

Ω_t 2

$\phi_t R_{nt} =$ 477.1k/connection

$R_{nt} / \Omega =$ 424.1k/connection

$T_u / \phi R_{nt} =$ 52.4%... OK

$T_a / (R_n / \Omega) =$ 47.2%... OK

4.8 Bolt Summary

Strength Limit States:

All strength limit states take the form:

LRFD	ASD
$R_u \leq \phi_t R_n$	$R_a \leq R_n / \Omega_t$
Req'd $R_n = R_u / \phi_t \leq R_n$	Req'd $R_n = R_a \Omega_t \leq R_n$
$R_u / (\phi_t R_n) \leq 1.00$	$R_a / (R_n / \Omega_t) \leq 1.00$

Which is: **FORCE on a bolt** \leq **STRENGTH of a bolt**

The STRENGTH of a bolt is computed by:

Simple Tension or Shear

Limit State	Specification	Nominal Capacity	Typical Design Variables	ϕ	Ω
Tensile Rupture	J3.6	Single Bolt Capacity: $F_{nt} A_b$	Bolt Material, Bolt Size	0.75	2.00
Shear Rupture	J3.6	Single Shear Plane: $F_{nv} A_b$	Bolt Material, Bolt Size	0.75	2.00
Slip Capacity	J3.8	Single Shear Plane: $\mu D_u h_f T_b$	Bolt Material, Bolt Size	1.00, 0.85, or 0.70	1.50, 1.76, or 2.14

Combined Shear and Tension:

Bearing Type Fasteners (-X or -N bolts):

- Modify the nominal tensile rupture capacity for the presence of shear (SCM J3.7)
- Apply the shear rupture limit state without modification

Limit State	Specification	Nominal Capacity, R_n	Typical Design Variables	ϕ	Ω
Tensile Rupture	J3.7	Single Bolt Capacity: $F_{nt} A_b$	Bolt Material, Bolt Size	0.75	2.00
Shear Rupture	J3.6	Single Shear Plane: $F_{nv} A_b$	Bolt Material, Bolt Size	0.75	2.00

Slip Critical Type Fasteners (-SC bolts):

- Modify the nominal slip capacity for the presence of tension (SCM J3.9)
- Apply the tensile rupture limit state without modification

Limit State	Specification	Nominal Capacity, R_n	Typical Design Variables	ϕ	Ω
Tensile Rupture	J3.6	Single Bolt Capacity: $F_{nt}A_b$	Bolt Material, Bolt Size	0.75	2.00
Slip Capacity	J3.9	Single Shear Plane: $\mu D_u h_f T_b k_s$	Bolt Material, Bolt Size	1.00, 0.85, or 0.70	1.50, 1.76, or 2.14

The FORCE on a bolt is computed by:

Forces Concentric with the Bolt Group at the Faying Surface:

- All bolts are assumed to be equally stressed in tension.
- All shear planes are assumed to be equally stressed in shear.

Eccentricity in the Plane of the Faying Surface:

- **Elastic Vector Method:** See SCM pg 7-8. Computes shear in the bolts. Direct method that is conservative and has an inconsistent factor of safety.
- **Instantaneous Center of Rotation Method:** See SCM pg 7-6. Computes the relationship between the applied load and the shear load in the worst case bolt. Iterative method that is more consistent with test results and not as conservative as the Elastic Method.

Eccentricity out of the Plane of the Faying Surface:

- **Case I Method:** See SCM pg 7-10. Basic mechanics (Mc/I) using the compression contact area to find the tension in the worst case bolt. Finding I_x may be iterative. If the shear is concentric with the bolt group it is equally divided among the shear planes otherwise use either the elastic vector or IC method to find the bolt shear forces.
- **Case II Method:** See SCM pg 7-12. Uses basic statics (Applied Moment = $P_e = r a t n'$ $d m =$ Internal Moment) without considering the contact area to find the tension in the worst case bolt. If the shear is concentric with the bolt group it is equally divided among the shear planes otherwise use either the elastic vector or IC method to find the bolt shear forces.

Types of Welds

The basic weld types are groove welds, fillet welds, and slot & plug welds.

Groove Welds

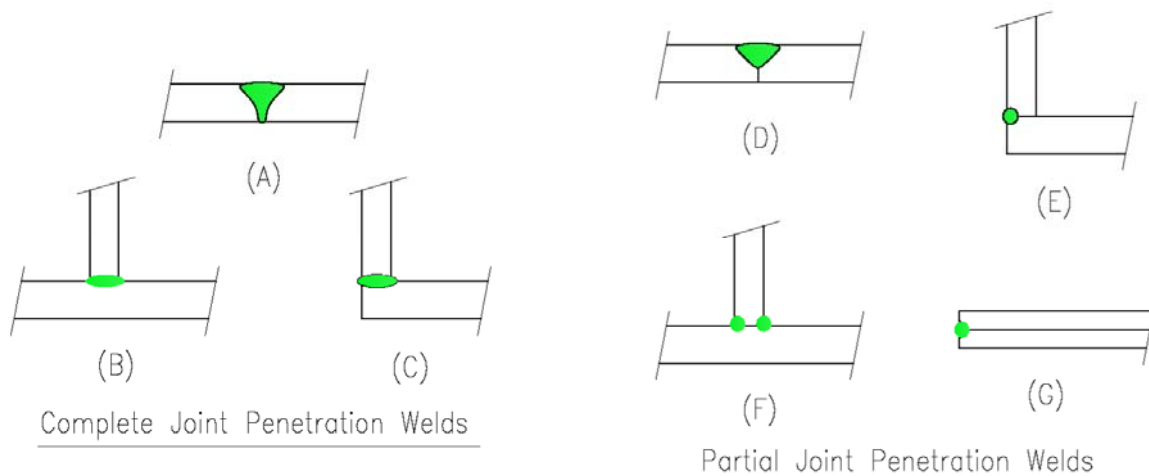
Groove welds are generally used to fill the gap between the two pieces being connected. They are called groove welds because the edges of the materials being joined are prepared so that there is a groove of some shape formed when the pieces are first laid together. The weld metal fills the groove.

Groove welds are considered to be either "**complete joint penetration**" (CJP) or "**partial joint penetration**" (PJP).

A CJP weld completely fills the gap between the two pieces. Parts A, B, and C of Figure 5.1.2 illustrate CJP welds. CJP welds made with appropriate filler material are stronger than the base metals that they connect, so strength calculations are not necessary.

A PJP weld only fills a portion of the gap as seen in Figure 5.1.2 parts D, E, F, and G. PJP welds are used when it is not required to develop the full strength of the connected parts to transfer the load.

Figure 5.1.2
Groove Weld Examples

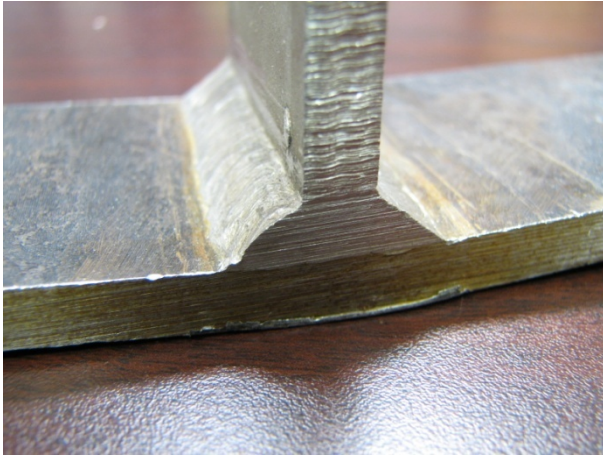


Fillet Welds

Fillet welds do not penetrate the gap between the parts being connected. A fillet weld generally has a triangular cross section with one leg of the triangle being attached to each piece being connected.

Fillet welds are very common and are used for a variety of connections. A typical fillet weld is shown in Figure 5.1.3.

Figure 5.1.3
Typical Fillet Welded T Joint



Slot & Plug Welds

Slot & Plug welds are similar to fillet welds in that they do not penetrate the gap between the parts being connected. These welds fill a slot or hole in one of the pieces being connected with the connection being between the edge of the slot or hole on the one piece and the surface of the other piece. The welds can be made in conjunction with fillet welds to shorten the lap of two pieces where space is limited.

Prequalified Welded Joints

The AWS specification defines a number of "prequalified" joints that can be made. Before a welded joint can be made on a project, it must be proven that the weld can be made using the desired materials and attain the required strength and ductility. Once the joint has been proven, a welding procedure that details how the weld is to be made is published and the procedure is considered to be prequalified. If the engineer specifies a joint or weld that has not been prequalified it is necessary for the welders to go through the qualification process to develop a new qualified welding procedure.

Before a welder is allowed to make a particular joint he/she must be CERTIFIED to make that weld. The certification process requires the welder to create the weld on a sample using the materials, procedure, and position that will be used for making the final connection. The sample is tested to insure that it meets specifications. Once a welder demonstrates that they can consistently create a weld that meets performance specifications then they are certified to make that particular weld.

The SCM Table 8-2 (SCM pages 8-34 through 8-64) presents that design parameters for the most common prequalified welds used for structural building connections.

Weld Symbols

A means for communicating the intent of the designer to the welder through standard weld symbols has been developed by the AWS. A table defining the weld symbols for prequalified welded joints is included in the SCM on page 8-35. You should take some time to examine this table. Pay particular notice to the notes at the bottom of the table. As an engineer you need to understand the language of the symbols or you may not get the weld that you are expecting.

Some things to notice:

1. The basic weld symbol consists of an arrow that points to the faying surface (i.e. the surface of contact between the pieces being connected) of the weld and a horizontal line where symbols are placed to describe the type of weld to be made.

The fillet, plug, and slot welds tend to transfer their forces via shear, so the predominate base metal failure is shear rupture. This is similar to the block shear limit state considered for tension member design.

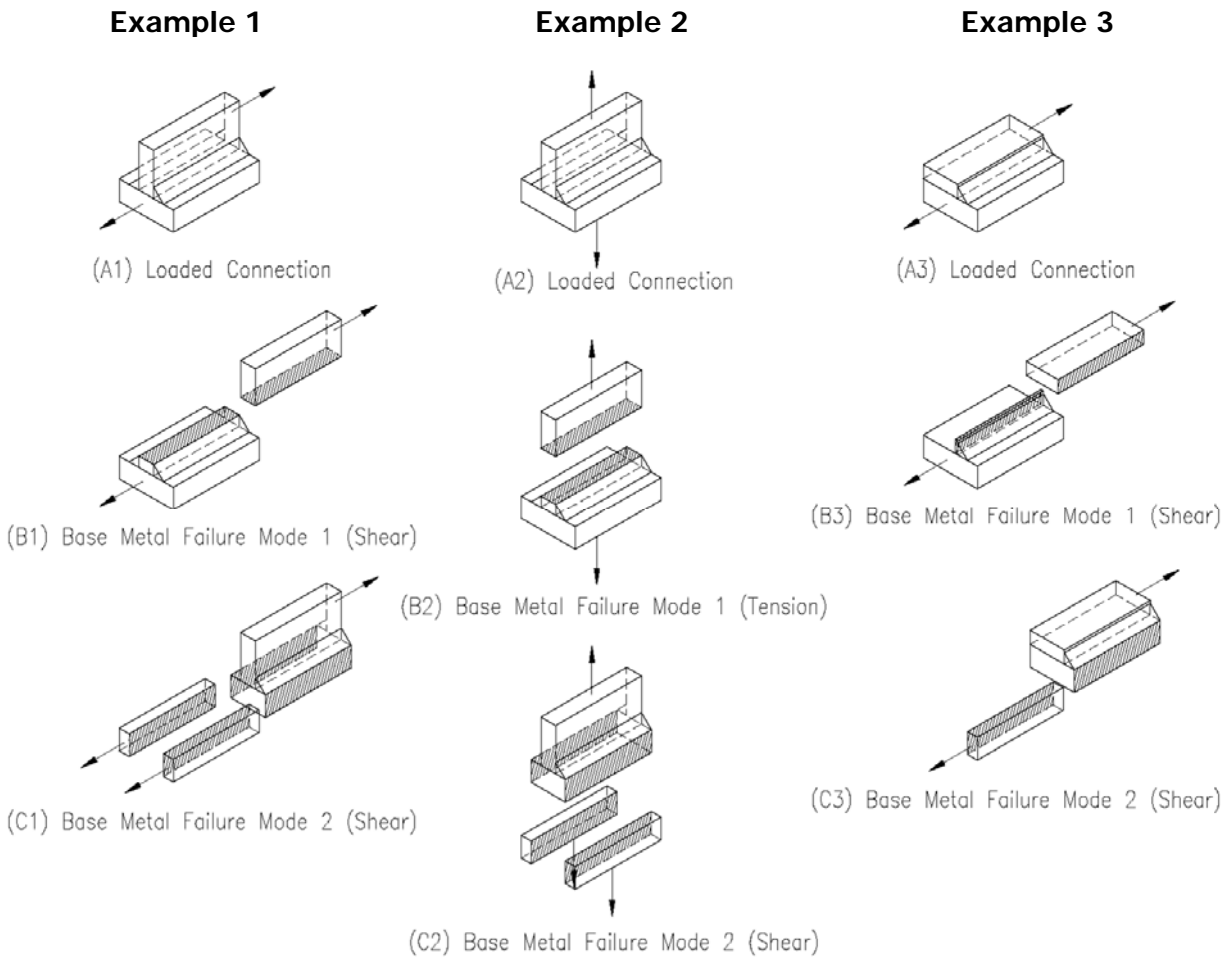
Base Metal Effective Area Example 1

Figure 5.4.1, example 1, shows a T connection made with two fillet welds (one on each side of the stem). The connection is loaded with a shearing force as shown.

As is the case in all connections, there are two potential base metal failure modes to consider. One for each base metal being joined. The effective areas (i.e. the failure surfaces) are shown with a hatch.

Failure mode 1 has an effective area equal to the thickness of the stem times the length of the weld. This failure surface supports the full shearing force. Also notice that this effective area "supports" two fillet welds. That is while the failure surface shown supports the full load, that load is transferred to two equal size fillet welds, each seeing half of the applied force.

**Figure 5.4.1
Base Metal Effective Area**



coplanar with the weld group, you never have the case of pure tension in the weld and consequently the coefficient on F_{EXX} will not reach 1.0. Figure 5.5.2 shows the variation of the coefficient with angle between weld axis and force direction.

This modified weld metal strength can be applied to any qualified segment of the weld. The criteria being:

1. The applied force is concentric with the weld group.
2. The applied force is in the plane of the faying surface (i.e. the applied force is in the plane of the weld group).
3. The weld is a fillet weld.
4. The weld group is a "linear weld group" as defined in the user note in SCM J2.4(a).

When the force is not concentric with the weld group and the IC method is used to determine the stresses in the weld group, the effect of the the load application being at an angle to the axis of the weld elements is covered in SCM J2.4(b). We presented the application of this method in BGSCM 5.2.

SCM J2.4(c) of the specification expands on SCM J2.4(a) and allows you to sum the capacities of the various weld length segments using SCM equation J2-10a (not using the SCM equation J2-5 modifier) to find the capacity of the weld group. Alternately SCM equation J2-10b is provided to approximate the same effect without computing the modified coefficient.

Largest Effective Fillet Weld Size

As previously mentioned in the effective area discussion, there is a point where increasing fillet weld size is ineffective because the base metal strength controls. This can be seen by looking at the strength equations.

Figure 5.5.3 shows an example where the weld is obviously stronger than the base metal. Adding additional weld to this connection would not have strengthened it. Additional weld would have been a waste of resources.

Consider the FBD in Figure 5.5.4 of a fractured weld and fractured base metal. This figure is taken from Figure 5.2.3.

We will first determine a relationship between weld size and base metal thickness for the condition shown in Figure 5.5.1. In this FBD the base metal force is shared between two equal size fillet welds (one weld on either side of the base metal). The largest effective fillet weld size will be the size where the weld strength equals the base metal strength:

$$2(0.6F_{EXX}) t_e L \geq (0.60F_u) t_{BM} L$$

**Figure 5.5.3
Base Metal Failure**



5.6 Designing Welds

The principle design variables for designing welds are:

- weld size (a for fillet welds),
- length (L_w),
- electrode type (F_{EXX}), and
- the layout of the weld.

Another factor that affects weld strength is the strength of the base metal. However, for purposes of this discussion we assume that the connected parts have already been chosen so the applicable thicknesses and material strengths are fixed.

It is possible to have multiple weld sizes and segment lengths a weld group. There are an infinite number of combinations of the design variables.

The objective of the design is to ensure that you have the least cost weld that satisfies the applicable limit states.

Selecting Fillet Weld Size

Fillet weld size selection is limited by both strength and stipulated limits in the specification. When selecting a leg size, a , you should determine three different numbers:

- Minimum ALLOWED size
- Maximum ALLOWED size
- Largest EFFECTIVE size

The limit MINIMUM ALLOWED SIZE was presented in BGSCM 5.3. The minimum allowed size is found in SCM Table J2.4.

The MAXIMUM ALLOWED SIZE is based on the geometric area available to accommodate the weld. The discussion on this topic can be found in Section 5.3.

If the weld metal's strength exceeds the base metal strength then increasing the weld's size further will not increase the strength of the connection. The weld size for which the weld base metal strength equals the base metal strength is known and the LARGEST EFFECTIVE size. This was discussed in BGSCM 5.3 and BGSC 5.5. Rounding up to the nearest 16th of an inch results in the largest effective weld size.

The weld size, a , then must satisfy the following criteria:

$$\text{min allowed size} \leq a \leq \text{min}[\text{max allowed size, largest effective size}]$$

When selecting the weld size, you should be aware that any fillet weld with a leg size, a , over 5/16" will require a minimum of three passes to complete. This adds substantially to the cost of the weld. If you have the choice (i.e. not limited by the above equations or by connection geometry) choose fillet welds that are not larger than 5/16".

5.9 Homework Problems

The homework problems involve the design of elements of three different structures plus some unrelated details. The drawing sets can be downloaded in PDF format from:

<http://bgstructuralengineering.com/HmwrkDwg/index.htm>

When completing the problems, consider both ASD and LRFD design philosophies unless otherwise specified by the instructor. Consider all limit states presented in this and prior chapters. Consider developing a generic spreadsheet that you can apply to similar problems.

Miscellaneous Steel Problems

Problems M5.1: MISCDET_STL_STL 1/S2.2 shows a braced pile system for a dock structure located in Whittier, Alaska. The drawing is not an accurate as-built drawing. Assume an E70XX electrode has been used. Express your results in terms of service load levels assuming that the applied load is 0.5 part dead load, 0.5 parts live load, and 1.75 part seismic load.

- **Problem M5.1a:** Determine the capacity of the weld that connects the diagonal brace to the gusset plate as shown in the detail. Consider both the strength of the weld and the base metal. Ignore any eccentricity.
- **Problem M5.1b:** Is this a "balanced" weld? If not, what is the eccentricity and how will it affect the capacity of the connection?

Problem M5.2: MISCDET_STL 2/S2.1 shows two MC sections attached to a gusset plate. Using a 1/2" thick gusset plate, a pair of C10x30 channels, E70XX electrodes and A588 steel design a fillet welded connection that is 50% stronger than the channels in tensile yielding. Neatly draw a scaled detail of the connection. *Note: There are a number of situations where parts of a structure are designed to be stronger than other parts, regardless of structural analysis results. This is typically done to ensure that a particular failure mode will not control the design. In this case, failure would be tensile yielding of the brace instead of fracture of the connection!*

Problem M5.3: For this problem refer to drawing MISCDET_STL 4/S2.1. A pair of L3x3x5/16 are welded to a 7/16" thick gusset plate with fillet welds as shown. Use E70XX electrodes. Design the connection to have a balanced weld group and to have a capacity 25% greater than the tensile yielding capacity of the angles.

Problems M5.4: For this problem refer to drawing MISCDET_STL 4/S5.1. Create a general purpose spreadsheet (in MS Excel unless otherwise approved by your instructor) to compute the capacity, P_n for the fillet welded connection shown. Consider only weld strength. The design variables are the weld leg size "a", electrode type and grade, the distances A and B, the location of the force (X), and the angle of loading (Angle) from the vertical. Group the design variables and the resulting value at the beginning of the spreadsheet.

- **Problem M5.4a:** Use the Elastic Vector method
- **Problem M5.4b:** Use the Instantaneous Center of Rotation method.

Problems M5.5: For this problem refer to drawing MISCDET_STL 5/S5.1. Create a general purpose spreadsheet (in MS Excel unless otherwise approved by your instructor) to compute the capacity, P_n for the fillet welded connection shown. Consider only weld strength. The design variables are the weld leg size "a", electrode type and grade, the distances A and B, and the location of the force (X). Group the design variables and the resulting value at the beginning of the spreadsheet.

Problem M5.6: Using the connection in MISCDET_STL 1/S1.3, assume that the W section is a W14x30 and the plates are all 3/8" thick. F7XX electrodes are used. Determine the minimum practical length "L" for which the weld connection is at least as strong as the bolted connection. Note that all welds are equally stressed since they deform equally.

Problem M5.7: Using the connection in MISCDET_STL 1/S1.3, assume that the W section is a W16x31, the plates are all 1/2" thick, and $L = 16"$. Determine the maximum moment nominal moment, M_n , that the welded connection can transfer. Consider that only the flange plates are effective in transferring moment. Recall that moment equals the couple formed by the weld forces on each flange times the distance between the center of the connection plates.

Dormitory Building Design Problems

The braces in this building may be connected to the frames by welds, depending on the detail used. In these problems, the connections will be designed based on weld strength and shear rupture of the base metal.

Problem D5.1: For a brace on the first floor level on Grid 2 and between Grids A & B determine the required size and length of fillet weld to carry the required force while minimizing the lap length, L, between the brace and gusset plate. Assume that the gusset plate is 7/16" thick. Neatly draw a scaled detail of the final connection.

- **Problem D5.1a:** Assume that the brace consists of a pair of angles (L4x4x3/8) and is connected to the frame by a connection similar to MISCDET_STL 4/S2.1.
- **Problem D5.1b:** Assume that the brace is an HSS 5x5x1/4 and is connected to the frame by a connection similar to MISCDET_STL 5/S2.1.

Chapter 6 – Buckling Concepts

6.1 Buckling Basics

Up to this point, we have dealt with members and components in tension. Tension has the benefit of causing a structural element to straighten. As we move the discussion to compression, we find that the strength of members may be compromised by *buckling*.

There are two main modes of buckling failure that may be experienced by steel members: Overall (or general) buckling and local buckling. Figure 6.1.1 illustrates the difference between the two modes.

General buckling is characterized by a distorted, or buckled, longitudinal axis of the member. In local buckling, the axis of the member is not distorted, but the strength of the cross section is compromised by the buckling of a component of the cross section.

As learned in mechanics, the Swiss mathematician Leonhard Euler developed an equation that predicts the critical buckling load for a straight pinned end column. The equation is:

$$P_{cr} = \pi^2 EI / L^2$$

Dividing by the area of the element, we get an equation for the critical buckling stress:

$$\sigma_{cr} = \pi^2 E / (L/r)^2$$

Where the member cross sectional dependent term (L/r) is referred to as the "slenderness" of the member.

Graphing Euler's equation shows that the predicted buckling stress gets quite large for small values of slenderness. At these levels, the critical buckling stress exceeds the capacity of the material to handle it (F_y), so the limiting strength becomes the material strength instead of buckling. The equation for maximum compressive stress then becomes:

$$\sigma_{max} = \text{minimum}[\pi^2 E / (L/r)^2, F_y]$$

This relationship is graphed in Figure 6.1.2.

Figure 6.1.1
General vs. Local Buckling

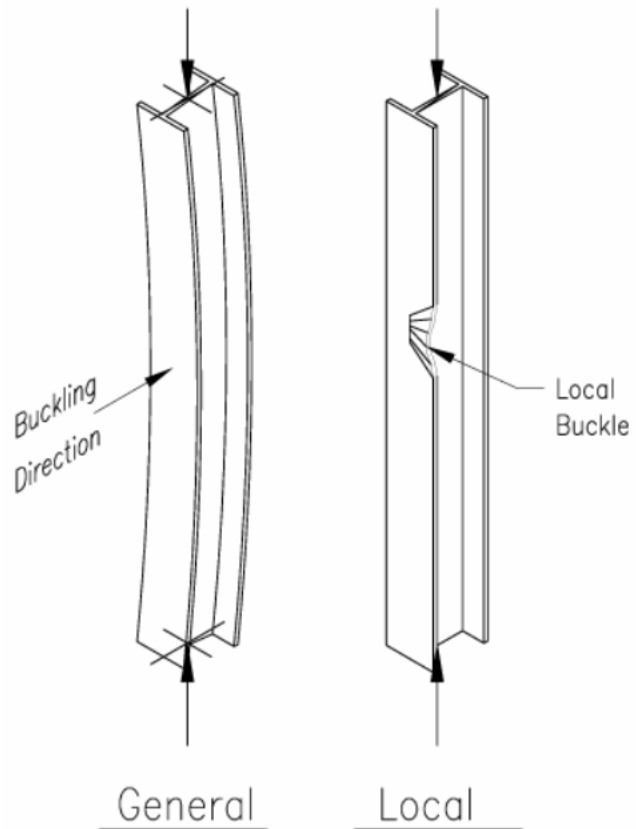
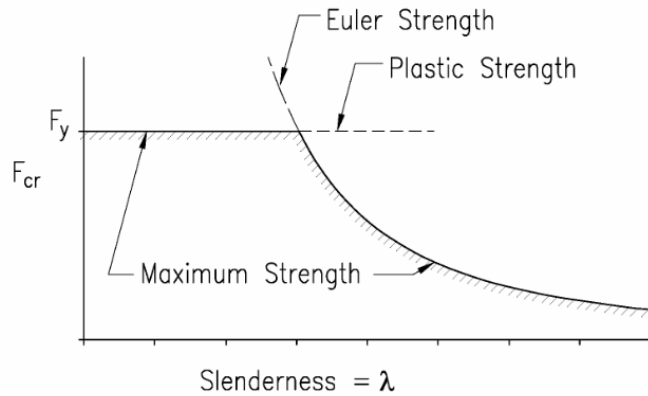


Figure 6.1.2
Theoretical Maximum Compressive Stress

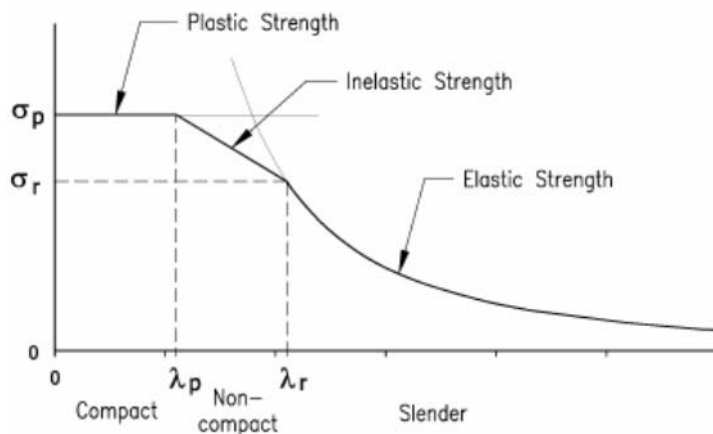


Experimental data shows that the above lines are UPPER limits of actual member strength. The data also shows that the transition from plastic (i.e. $\sigma_{max} = F_y$) behavior to elastic ($\sigma_{max} = \text{Euler stress}$) is not as clean as shown above. The transition contains characteristics of both plastic and elastic behavior. As a result the transition range is often referred to as the "inelastic" range.

Several methods for approximating strength in the inelastic range have been used for different materials over the years. However, for purposes of discussion, a linear transition will be used here.

In the SCM slenderness values at the transition points are known as λ_p and λ_r . These values are shown on Figure 6.1.3. The strength values associated with these points we will call σ_p and σ_r . (Note that you will not find these terms in the SCM.)

Figure 6.1.3
Theoretical Maximum Compressive Stress



The graph in Figure 6.1.3 is the typical strength curve associated with both general and local buckling behavior.

As can be seen in Figure 6.1.3, there are three distinct regions on the strength curve when buckling is a possibility. In the case of **general buckling** the ranges are known as the PLASTIC, INELASTIC, and ELASTIC ranges. When the member slenderness (L/r) is low then plastic behavior is expected and the member strength is limited by plastic strength of the material σ_p . When the member slenderness is in the

transition range, then inelastic behavior and member strength is a combination of plastic and elastic strength. Finally, when member slenderness is large then elastic buckling behavior is expected and the strength of the member is controlled by Euler buckling behavior.

For the case of **local buckling** (as discussed in the next section) the slenderness is based on width/thickness ratios of the slender plate elements that make up the cross section of most steel members. The member cross sections are then classified by which of the three ranges their most slender element falls in. If the most slender cross sectional element is not very

Basically, the SCM method for computing member slenderness (SCM Appendix 7) for partial end rotational restraint computes a relative stiffness factor, "G", at each end of the column segment that accounts for the stiffness of all members that contribute to rotational resistance in the plane of the frame at the connection. You only consider members that have moment connections to the joint in the plane of buckling being considered.

A powerpoint presentation that illustrates the SCM method for determining effective length coefficients can be found on the web page for this text section on the text website.

Using this method, the effective length for most columns can be determined. Example problem 6.3 shows how the method is applied to columns in a hypothetical frame.

Having the ability to determine effective lengths, we can now determine the strength of a column, however we also need to take a look at local buckling.

6.3 Local Buckling

Figure 6.3.1
Flange Local Buckling Example



The cross sections of steel shapes tend to consist of an assembly of thin plates. When the cross section of a steel shape is subjected to large compressive stresses, the thin plates that make up the cross section may buckle before the full strength of the member is attained if the thin plates are too slender. When a cross sectional element fails in buckling, then the member capacity is reached. Consequently, local buckling becomes a limit state for the strength of steel shapes subjected to compressive stress.

Figure 6.3.1 shows an example of flange local buckling. This member failed before the full strength of the member was realized because the slender flange plate buckled first.

In the Euler equation the parameter (L/r) is known as the slenderness of the member. For a plate, the slenderness parameter is a function of the width/thickness (b/t) ratio of a slender plate cross sectional element. In the SCM *member cross sections* are classified, as introduced in the prior section, as being COMPACT, NON-COMPACT, or SLENDER based on the width/thickness ratios of the plate elements that make up their cross sections. All cross sections can be classified solely on cross sectional dimensions and type of steel being used.

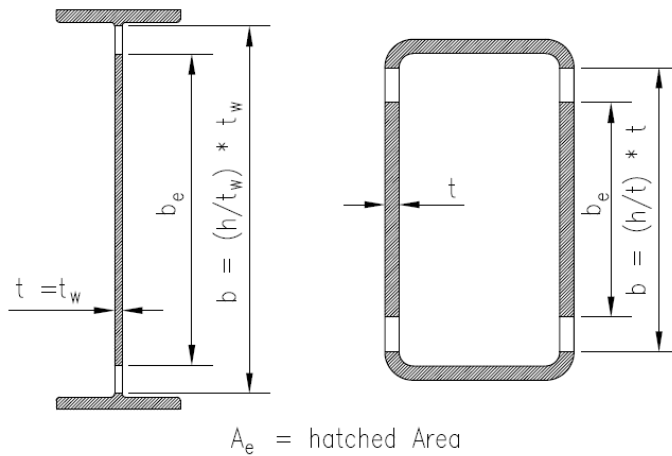
Members with plates having b/t ratios between 0 and λ_p are said to be "compact" and can be expected to not exhibit any buckling behavior. Their compressive strength is limit by the material compressive strength as seen in Figure 6.1.3.

Members with plates having b/t ratios between λ_p and λ_r are said to be "non-compact" and exhibit both plastic and elastic buckling behavior. Their compressive strength is limited by inelastic buckling.

For example in a W section, the stiffened cross sectional element is the web. The web has a "width, b " = $(h/t_w) * t_w$, where both h/t_w and t_w are tabulated values. The effective width, b_e , is found with SCM equation E7-17. The effective width, b_e will always be taken as less than or equal to width b . A_e is the gross area, A_g , less *the area lost* using the reduced web plate width. When b_e is less than b , the loss in cross sectional area becomes $(b - b_e)*t_w$. The resulting A_e equals $A_g - (b - b_e)*t_w$.

From a buckling perspective, the critical buckling stress is achieved when either local buckling capacity or general buckling capacity is reached. If a *slender section* is part of a *slender member* then, as the general slenderness parameter (KL/r) increases, there is a point where general buckling will occur before local buckling will occur. If this happens, then local buckling is a moot point and there is no need to adjust the member capacity for the effects of local buckling. The SCM equations in E7.2a do this. The equations are a function of the general buckling critical stress which is a function of general slenderness (KL/r) . The result is that there are many times when a section has slender cross sectional elements, but there is no reduction in capacity as a result and $Q_s=1$. In other words, cross sectional element slenderness does not always impact member capacity.

Figure 7.4.2
 A_e for Slender Sections



For "box" sections, you typically have two slender elements at a time. This means that you have two cross sectional elements that are losing area. Both need to be accounted for in your A_e calculation.

Figure 7.4.2 illustrates A_e on a W section and rectangular HSS.

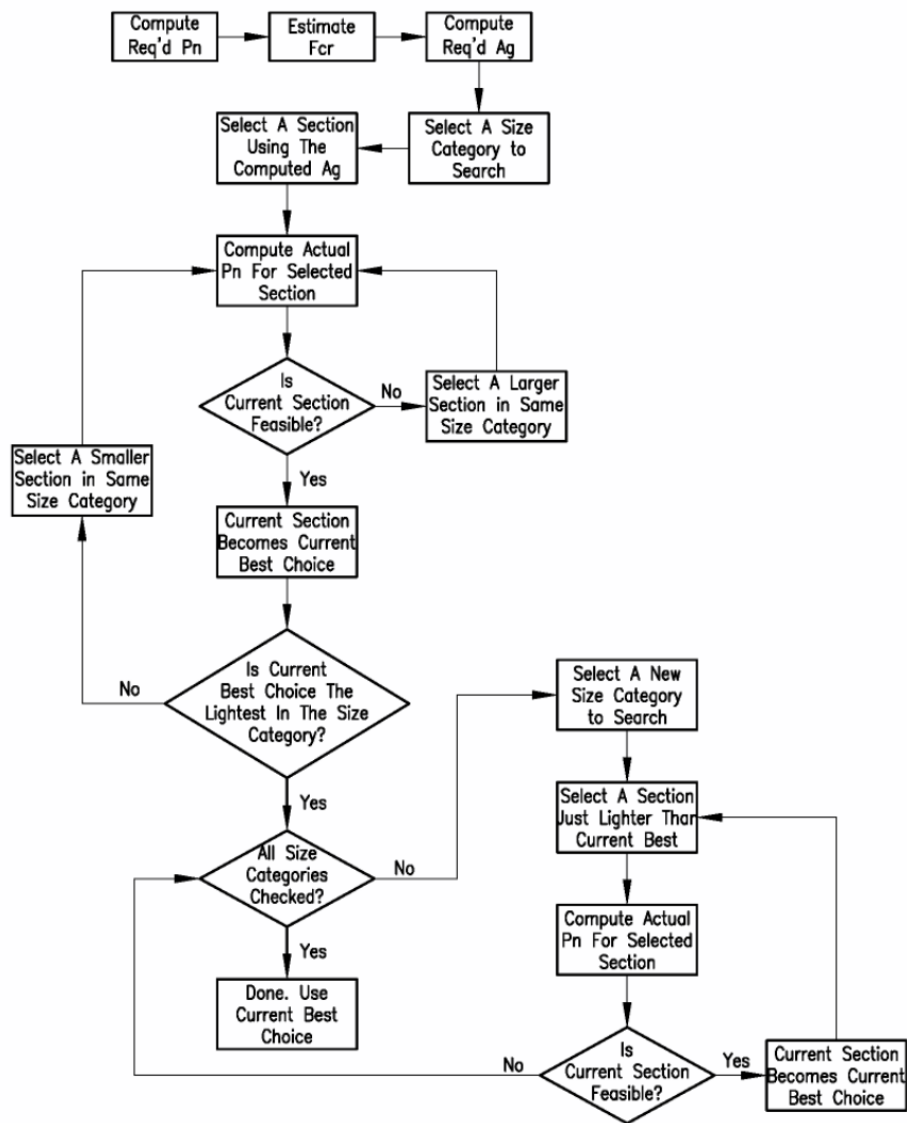
Note the gray box on SCM page 16-1.43 that allows you to use F_y for f when using SCM E7.2b. This makes the computation much easier, is slightly conservative, and in few cases does the extra amount make a significant difference.

Sample Spreadsheet Calculation

The following spreadsheet example computes the nominal capacity of a W section column according to the requirements of SCM E7. The local buckling cases and computation of Q_s and Q_a change if you go to other types of compression members.

Try	W12X30
A_g	8.79 in ²
r_x	5.21 in
r_y	1.52 in
$b_f/2t_f$	7.41
h/t_w	41.8
t_w	0.26 in

Figure 7.6.1
Flow Chart of Selection Procedure

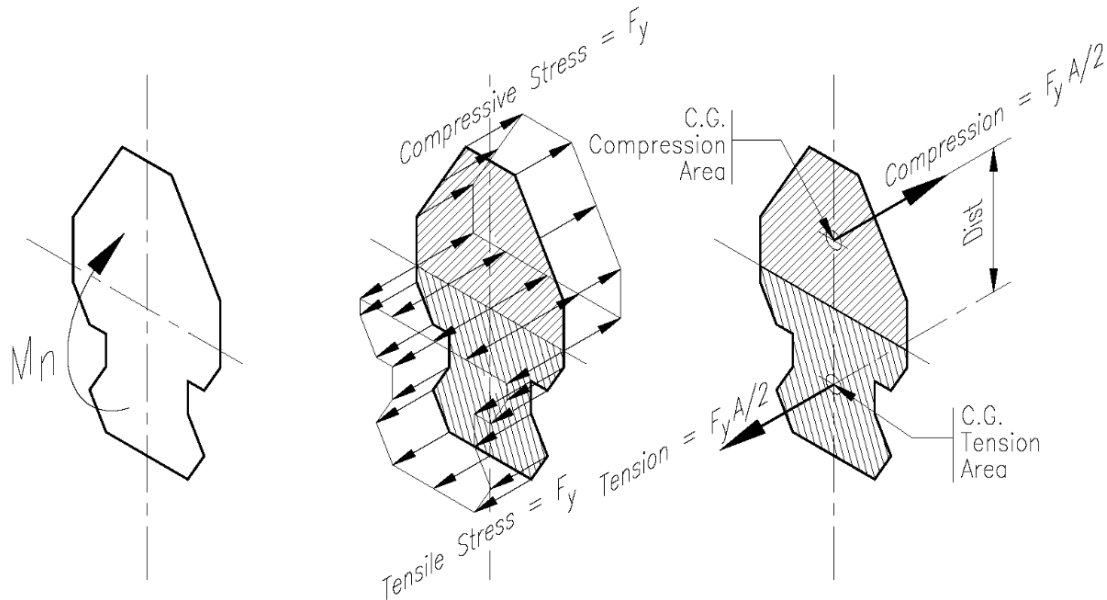


You can also use this method by manually entering the associated section properties manually (or by judicious use of the spreadsheet steel tables available from the AISC website) into an analysis spreadsheet of your own making.

Computer Methods

There are probably a thousand different approaches that you can take to search the section tables if you are adept at using spreadsheets. Even more efficient would be making use of computer programming either within or outside of your spreadsheet to conduct the search. Most spreadsheet programs incorporate Visual Basic for Applications (VBA) or some other language that makes programming search algorithms possible and is not too difficult.

Figure 8.2.1.3
 M_n Determination



$$M_n = F_y * (A_g/2) * (\text{dist between centroids of the half areas})$$

$$M_n = F_y * [(A_g/2) * (\text{dist between centroids of the half areas})]$$

The term within the square brackets is a section property since it is wholly dependent on cross sectional geometry. This section property, in the SCM, is referred to as the plastic section modulus and is given the symbol "Z".

$$Z = (A_g/2) * (\text{dist between centroids of the half areas})$$

Being a section property, this term is tabulated for all SCM shapes in the section property tables. Note that the value of Z is axis dependent. Thus, the value of Z about each principle axis is tabulated in the tables.

For sections not in the tables (for example, built up sections), the value of Z is readily determined using the definition of Z. A procedure might be:

- determine A_g
- Locate the centroidal axis
- Locate the centers of the two half areas
- Compute Z

If a section is doubly symmetric, this process can be shortened by taking the first moments of area for half of the section. Another approach would be to use the convenient SCM Table 17-27 (SCM pg 17-36) which tabulates properties of some common geometric shapes. Values of Z, like values of I, are additive if taken about the same axis, so adding values of Z for symmetric components of a cross section is an option.

Table 8.2.2.1
Limit States and Applicable Chapter F Sections
For Doubly Symmetric I-Shapes (W, M, S, HP and built-up Sections)
Bent About Their Major Axis

Flange	Web	Limit State				
		Y	CFY	LTB	FLB	TFY
Compact	Compact	F2.1	----	F2.2	----	----
Compact	Non-Compact	----	F4.1	F4.2	F4.3a	F4.4
Compact	Slender	----	F5.1	F5.2	F5.3a	F5.4
Non-Compact	Compact	----	----	F3.1	F3.2a	----
Non-Compact	Non-Compact	----	F4.1	F4.2	F4.3b	F4.4
Non-Compact	Slender	----	F5.1	F5.2	F5.3b	F5.4
Slender	Compact	----	----	F3.1	F3.2b	----
Slender	Non-Compact	----	F4.1	F4.2	F4.3c	F4.4
Slender	Slender	----	F5.1	F5.2	F5.3c	F5.4

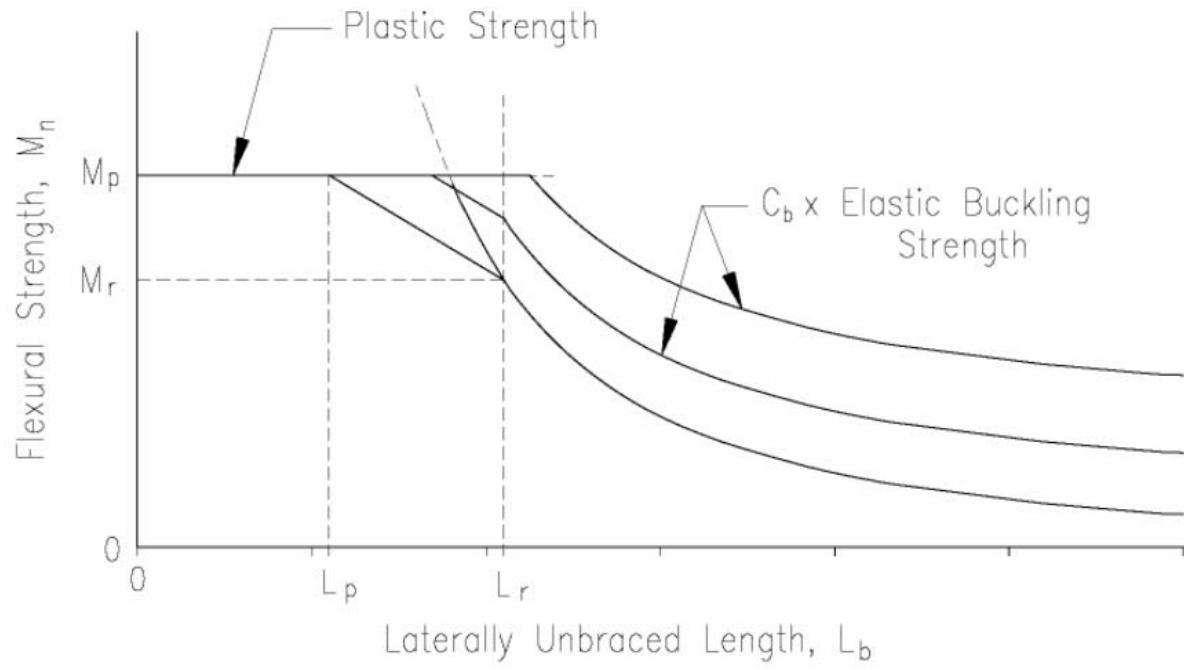
Table 8.2.2.2
Limit States and Applicable Chapter F Sections
For Singly Symmetric I-Shapes with Web Attached at Mid Flange
Bent About Their Major Axis

Flange	Web	Limit State			
		CFY	LTB	FLB	TFY
Compact	Compact	F4.1	F4.2	F4.3a	F4.4
Compact	Non-Compact	F4.1	F4.2	F4.3a	F4.4
Compact	Slender	F5.1	F5.2	F5.3a	F5.4
Non-Compact	Compact	F4.1	F4.2	F4.3b	F4.4
Non-Compact	Non-Compact	F4.1	F4.2	F4.3b	F4.4
Non-Compact	Slender	F5.1	F5.2	F5.3b	F5.4
Slender	Compact	F4.1	F4.2	F4.3c	F4.4
Slender	Non-Compact	F4.1	F4.2	F4.3c	F4.4
Slender	Slender	F5.1	F5.2	F5.3c	F5.4

Table 8.2.4.1
Lateral Torsional Buckling Limit State Summary

SCM Section	L_p	L_r	M_p	M_r	M_n , Inelastic Range $L_p < L_b \leq L_r$	M_n , Elastic Range $L_r < L_b$
F2 & F3	Eq F2-5	Eq F2-6	$F_y Z_x$	$0.70 F_y S_x$	Min[$C_b(M_p - (M_p - M_r)(\lambda - \lambda_p)/(\lambda_r - \lambda_p))$, M_p]	Eq F2-3 or F2-4
F4	Eq F4-7	Eq F4-8	$R_{pc} F_y S_{xc}$	$F_L S_{xc}$		Eq F5-4 or F5-9
F5	Eq F4-7	Eq F5-5	$R_{pc} F_y S_{xc}$	$R_{pc}(0.7F_y)S_y$		
F9	----	----	Eq F9-2 or F9-3	----	Eq F9-4	

Where: L_b = Laterally Unbraced Length



shear. This propensity to buckle is a function of the thickness of the plate and dimensions of the shear panel.

Figure 8.3.1.2 shows a typical strength curve based on slenderness.

The Effect of Stiffening Shear Panels

Shear strength can be improved by adding plate stiffeners to prevent out-of-plane buckling of the web. These stiffeners can be spaced to reduce the shear panel dimensions so that failure is plastic deformation instead of elastic buckling. Figure 8.3.1.3 shows how these stiffeners are arranged.

The specification includes provisions for strengthening of shear panels with stiffeners. SCM G2.2 requires that the moment of inertia about an axis centered in the beam web meet certain criteria. We go into much more detail on the design of these elements in a later section.

8.3.2 Shear Strength Limit State

The computation for the shear strength of steel sections is found in SCM Chapter G. Section G2 investigates the pre-buckling strength of stiffened and unstiffened webs. Section G3 considers post buckling strength (or the strength related to tension field action). This text focuses of section G2.

The Limit State

The basic limit state follows the standard form. The statement of the limit states and the associated reduction factor and factor of safety are given here:

Figure 8.3.1.2
Shear Strength as a Function of Plate Slenderness

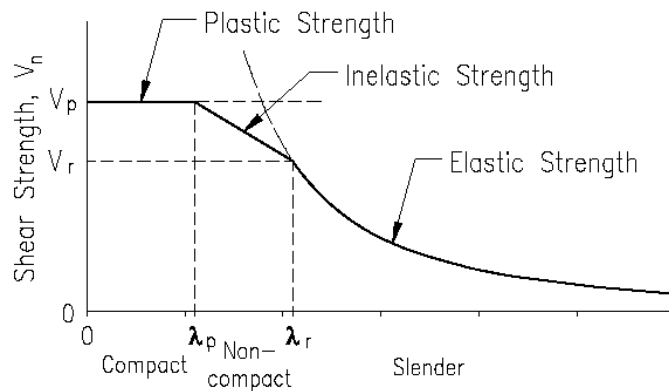


Figure 8.3.1.3
Transverse Web Stiffeners

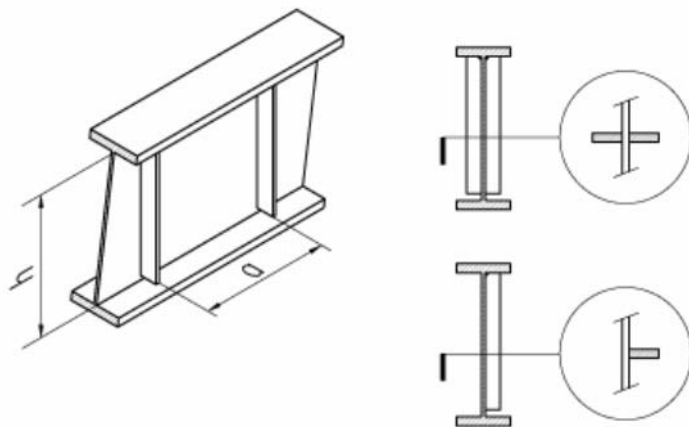
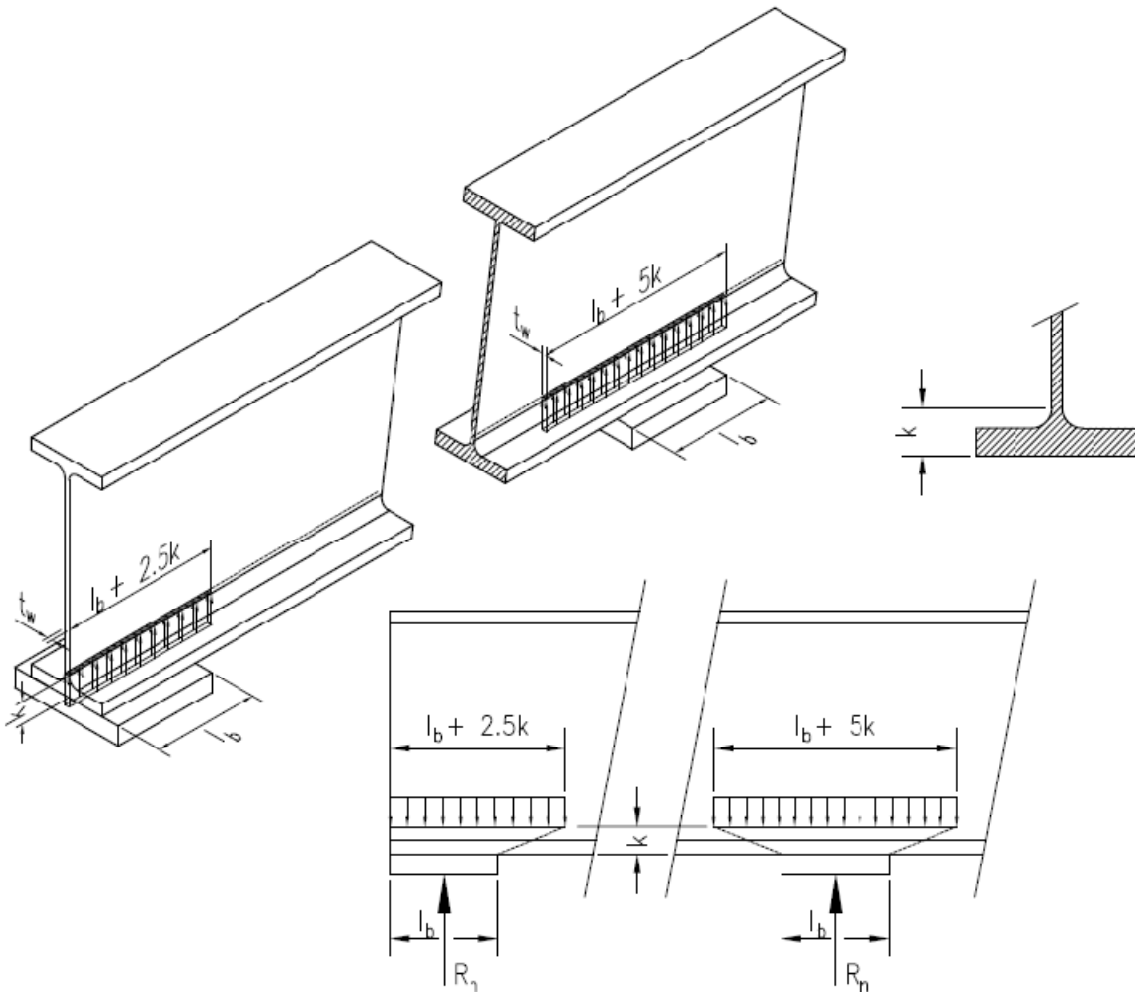


Figure 8.5.1.1
Web Local Yielding Parameters



Note that, at the end of the beam, there is less material available in the web (i.e. the length of the critical area is shorter) than when away from the end of the beam. Consequently there is not as much strength available near the end of the beam as there is when away from the end of the beam.

The maximum stress that can occur on this area is taken to be the steel yield stress, F_y . Thus the maximum force that can be applied equals the critical area times the yield stress.

As a designer, once a beam is chosen and the reaction is known, the only variable you have direct control over (without adding bearing stiffeners) is the actual bearing length N .

Bearing stiffeners, similar to shear stiffeners except they are in contact with the flange where the load is applied, distribute the shear reaction over the depth of the web. The critical area can be increased by the addition of bearing stiffeners, which are discussed in a different section of this text.

The Limit State

SCM J10.2 covers web local yielding due to concentrated point loads applied to the flange.

The basic limit state follows the standard form. The statement of the limit states and the associated reduction factor and factor of safety are given here:

LRFD	ASD
$R_u \leq \phi R_n$	$R_a \leq R_n / \Omega$
Req'd $R_n = R_u / \phi \leq R_n$	Req'd $R_n = R_a \quad \Omega \leq R_n$
$R_u / (\phi R_n) \leq 1.00$	$R_a / (R_n / \Omega) \leq 1.00$
$\phi = 1.00$	$\Omega = 1.50$

The values of R_u and R_a are the LRFD and ASD factored loads, respectively, applied to the beam.

In this case R_n is the nominal web yielding strength of the member is computed using SCM equations J10-2 and J10-3 re-arranged here slightly:

$$R_n = F_y A_{cr}$$

Where:

- F_y is the tensile yield stress of the beam steel.
- A_{cr} is the critical cross sectional area at the top of the fillet that connects the web and flange of the member.
 - For forces applied away from the end of the beam, $A_{cr} = (5k + l_b) t_w$
 - For forces applied at the end of the beam, $A_{cr} = (2.5k + l_b) t_w$
- l_b is the actual bearing length on the flange. This is the length of contact with the supporting or supported element.
- k is the distance from the face of the flange to the top of the fillet that connects the web to the flange. This value is tabulated in the W shape section property tables.

This limit state is to be checked at each location where a concentrated force is applied transverse to the axis of a member. This typically occurs at support reaction locations and where the beam supports a column or beam reaction applied to the top of the top flange.

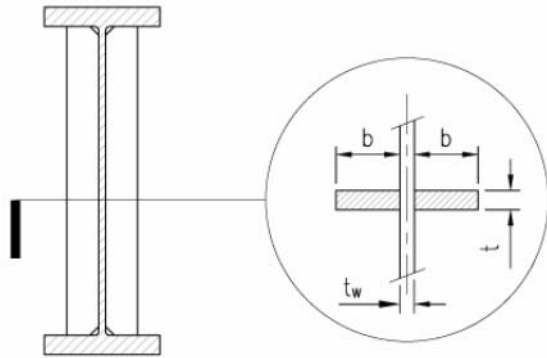
Sample Spreadsheet Calculation

The given spreadsheet example computes the reaction capacity, R_n , and controlled by local web yielding for a typical W section. The input values are in the grey shaded cells and the answer is where indicated.

Web Local Yielding		
SCM J10.2		
Section	W12x65	
Steel:	A992	
F_y	50	ksi
l_b	5	in
Tabulated Section Properties		
t_w	0.39	in
k	1.2	in
Location	End	Mid
A_e	3.12	4.29
R_n	156	214.5
$k \leftarrow$ Answer		

Stiffener plates can be applied to either just one side of the web or both sides. In either case the stiffeners must provide sufficient moment of inertia to prevent out of plane buckling. Figures 8.6.3.2 and 8.6.3.3 show the dimensions associated with each case.

**Figure 8.6.3.2
Transverse Stiffeners on Both Sides**

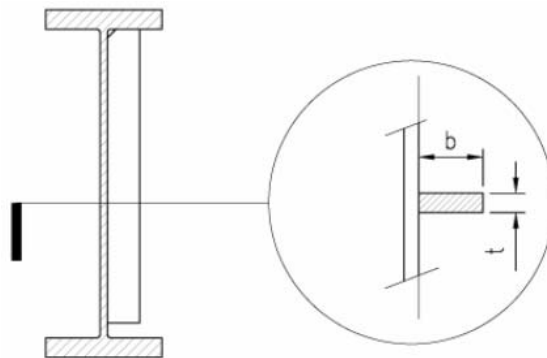


The stiffeners are typically steel plates welded to the web as shown in the figures. However, it is possible to bolt angles and other shapes to the web to accomplish the same purpose. In these cases basic principles of statics are used to compute the moments of inertia for the attached elements.

The goal is to provide more stiffener plate with a moment of inertia greater than or equal to what is required. Mathematically the goal is expressed as:

$$I_{st} \geq I_{req'd}$$

**Figure 8.6.3.3
Transverse Stiffener on One Side**



Determining the Required Moment of Inertia

The required moment of inertia to be provided by the transverse stiffener plates is given in SCM G2.2 as:

$$I_{req'd} = \min[a, h] t_w^3 j$$

Where:

- a is the spacing of the transverse stiffeners (see Figure 8.6.3.1)
- h is the height of the web--the distance between fillets
- t_w is the thickness of the web being stiffened
- j is a term whose value is determined using SCM equation G2-8. This term can be written as:

$$j = \max[0.5, 2.5 / (a/h)^2 - 2]$$

Determining the Actual Moment of Inertia

SCM G2.2 states that the moment of inertias are computed about an axis at the center of the web for double stiffeners or at the face of the web for single stiffeners. This is done for plate stiffeners with the following equations:

In composite steel-concrete beams, the connection between the steel beam and concrete slab is typically accomplished by welding an object to the top flange of the beam that engages the concrete directly. This is most frequently done with steel studs that are quickly and easily welded to the beam. The studs can be seen in Figure 10.2.2.

Other methods of shear connection include small channels welded transverse to the beam or a spiral coil of small diameter rod welded to the top flange.

Since the concrete is generally at an extremity of the beam, it is ineffective in resisting beam shear. As a result, the presence of composite action is ignored when computing the shear capacity of the beam.

**Figure 10.2.2
Typical Welded Studs**



Computation of the nominal flexural strength, M_n , of the beam is computed either using a strength method or an elastic method. If the steel section's web is sufficiently compact (and all the rolled sections in the inventory are compact for $F_y \leq 50$ ksi) then strength methods are used. For plate girders (i.e. built up members) with slender webs, elastic stress distribution methods are used.

For strength methods, all the steel is assumed to be fully yielded (compression above the plastic neutral axis and tension below the plastic neutral axis) and the concrete is at its ultimate strength in the compression zone. The nominal moment capacity, M_n , is taken as the internal moment that results from the forces on the section components.

For the elastic method, elastic stresses are computed using the bending stress equation (Mc/I) on a transformed section.

This chapter focuses primarily on the strength method.

10.3 Shear Strength

Since the concrete does not contribute to the shear strength of the composite beam, SCM I4.2 states that "The available shear strength ... shall be determined based upon the properties of the steel section alone in accordance with Chapter G." While the section addresses composite beams with steel deck, it is the only section which looks at shear in composite beams. It is consistent with past practice to apply SCM G to the computation of shear capacity in composite beams.

Figure 10.4.2
 y_{PNA} is in the Concrete

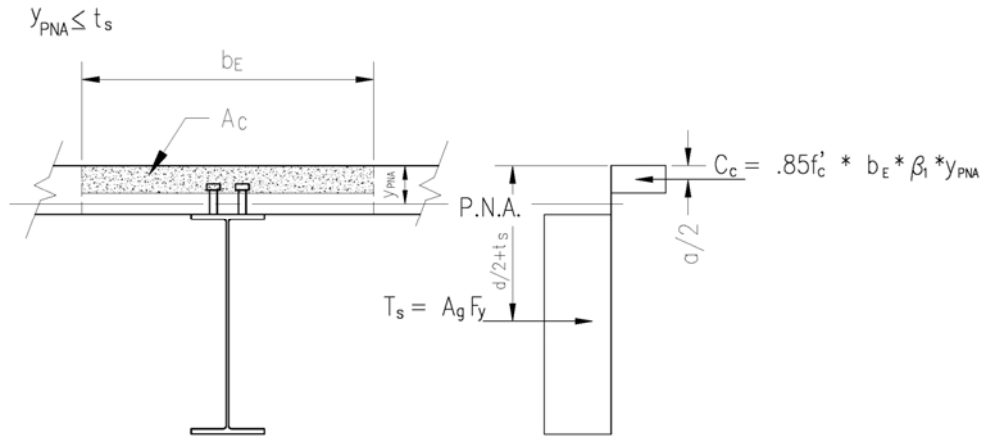


Figure 10.4.3
 y_{PNA} is in the Beam Flange

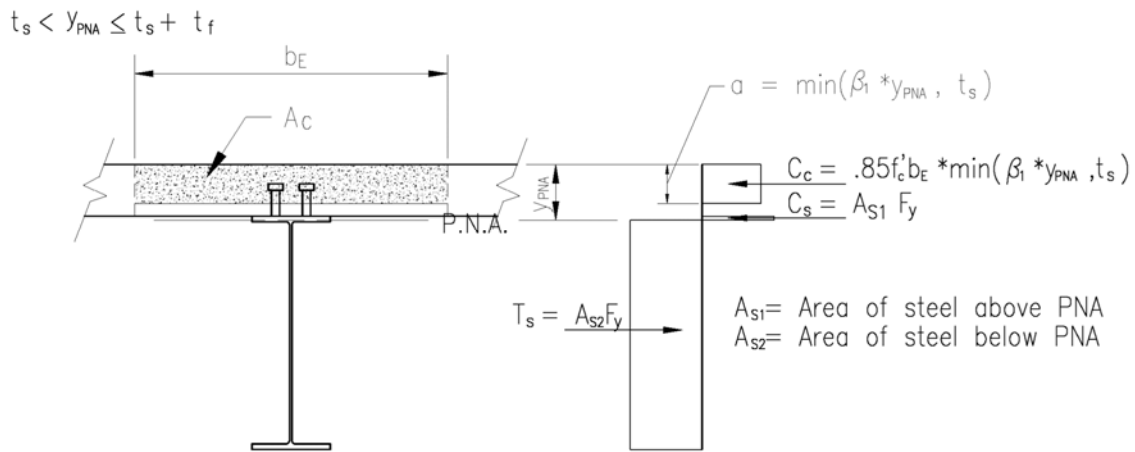
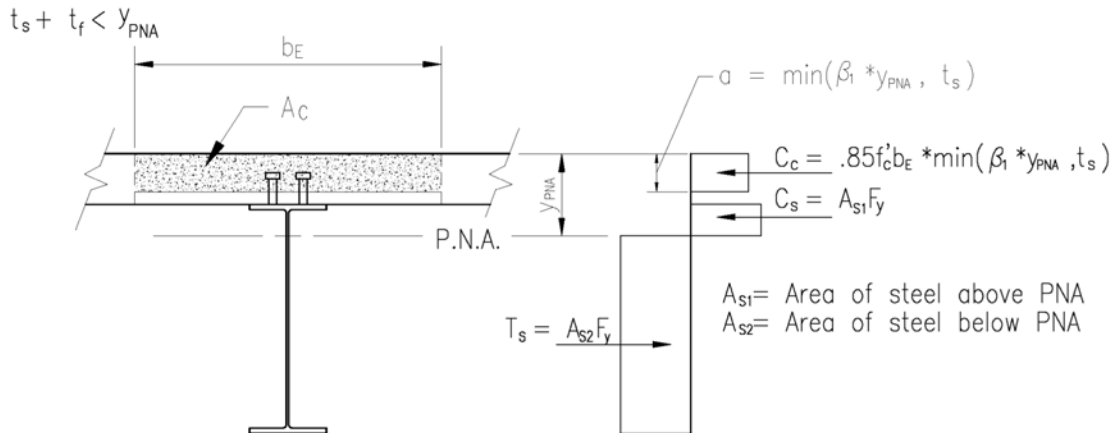


Figure 10.4.4
 y_{PNA} is in the Beam Web



References

The reference list presented here is, admittedly, very brief. The items that made the list are generally those books and articles that were actually used and/or quoted while producing this text. This is not to say that the information presented in the text is based on the author's research. It is far from it. The specifications, theory, and practices used in the profession come from a wealth of research done over the centuries by countless competent engineers, scientists, and mathematicians. While other texts have taken the time to search out the core documents that represent the current practices this one, unfortunately, has not.

With that said, here are the few references directly used and/or quoted in this text.

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